CHAPTER D-6 INTERNAL EROSION RISKS FOR EMBANKMENTS AND FOUNDATIONS

D-6.1 Key Concepts

One of the leading causes of dam and levee failures in the United States is from internal erosion of embankments (or their foundations). Unfortunately, this is a potential failure mode that cannot be completely analyzed using numerical formulae or models. However, valuable information on embankment and foundation characteristics and behavior is available to help in assessing internal erosion risks. The term "internal erosion" is used as a generic term to describe erosion of soil particles by water passing through a body of soil. "Piping" is often used generically in the literature but refers to a specific internal erosion mechanism (described below in the section on backward erosion piping) in this document.

D-6.2 Historical Background

Based on the records of dam incidents and the dam register in ICOLD (1974, 1983, 1995), Foster et al. (1998, 2000) evaluated the statistics of failure of large dams constructed between 1800 and 1986, excluding dams constructed in Japan before 1930 and in China. Approximately one-half of the cases of failure in operation were the result of internal erosion. The results are summarized in Table D-6-A-1 in Appendix D-6-A by the location of where internal erosion occurred. In this evaluation the three locations used were through the embankment, through the foundation, and from the embankment into the foundation. The largest number of internal erosion failures occurred through the embankment, and nearly one-half of these were associated with conduits or walls which penetrate the embankment. Approximately two-thirds of all failures and one-half of all accidents[1] occurred on first-filling or in the first 5 years of reservoir operation. Therefore, approximately one-half of all incidents included in this evaluation occurred after 5 years of

[1] Failure is collapse or movement of the dam or foundations such that water retention capability is lost, an accident is an event that was prevented from becoming a failure by remedial measures, and an incident is either a failure or accident (ICOLD 1974).

reservoir operation. Foster et.at. (1998, 2000) also found that nearly all internal erosion failures located through the embankment occurred when the reservoir level was at or near (within one meter) the pool of record. Excluding conduits and spillways, 63 percent of the incidents were associated with cracking, and 37 percent were associated with poorly compacted and high permeability zones (Foster et al. 1998, 2000). Additional statistics from this study are provided in Appendix D-6-A, including:

- The historical frequencies of failures and accidents (Table D-6-A-2).
- The timing of the incidents for internal erosion through the embankment (Table D-6-A-3) and for internal erosion through the foundation (Table D-6-A-4).
- A further assessment of the case study information for incidents of cracking and hydraulic fracturing in the embankment (Table D-6-A-5) and for incidents of poorly compacted and high permeability zones (Table D-6-A-6).

D-6.3 Physical Location of Internal Erosion Categories

As mentioned above internal erosion cases have been grouped into the following general categories related to the physical location of the internal erosion pathways in order to further evaluate internal erosion failure modes:

- Internal erosion through the embankment (Figure D-6-1)
- Internal erosion through the foundation (Figure D-6-2)
- Internal erosion of the embankment into the foundation (Figure D-6-3a), including along the embankment-foundation contact (Figure D-6-3b)

In addition to the general categories listed above, others added the following two locations when developing statistics from their internal erosion case history database (Engemoen 2017), as described in Appendix D-6-B:

- Internal erosion along or into embedded structures such as conduits or spillway walls (Figure D-6-4)
- Internal erosion into drains such as toe drains, stilling basin underdrains, etc.

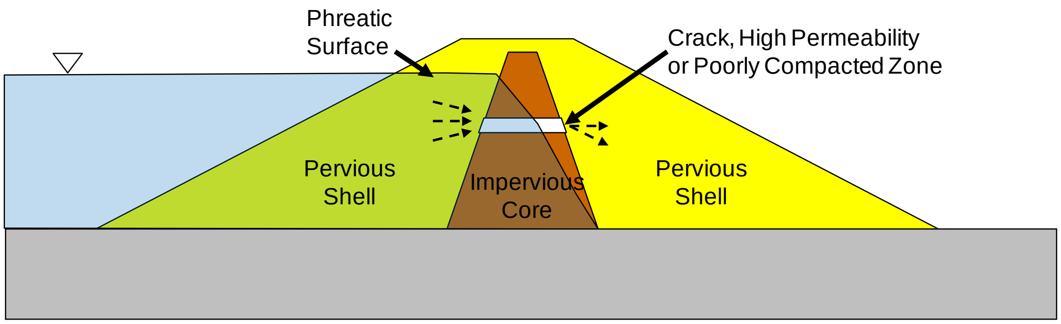


Figure D-6-1 Internal Erosion through the Embankment

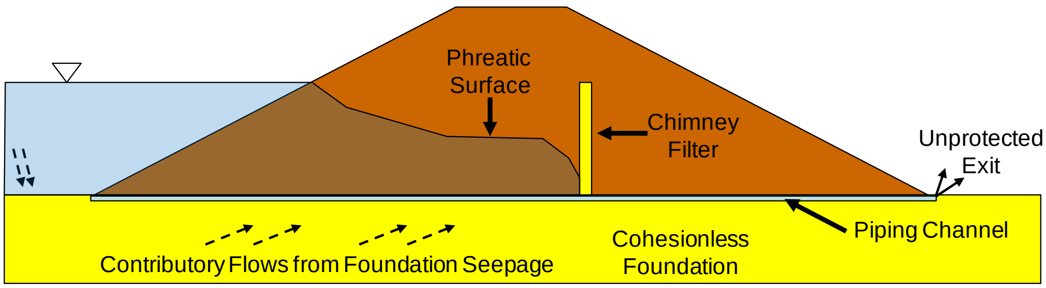


Figure D-6-2 Internal Erosion through the Foundation

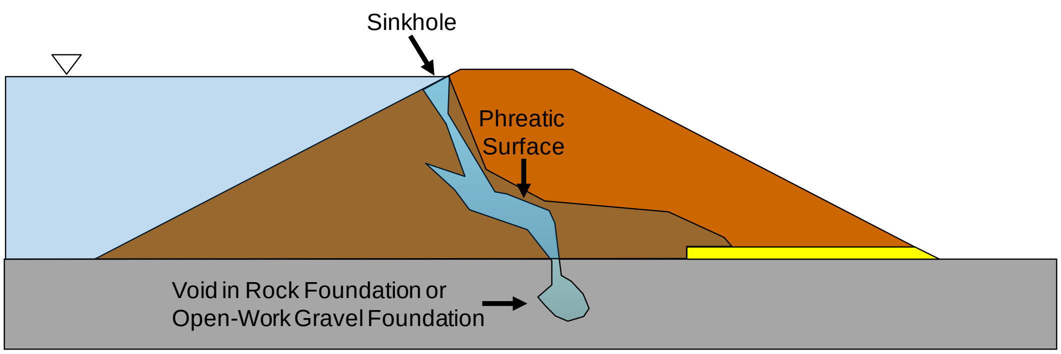


Figure D-6-3a Internal Erosion of the Embankment into the Foundation

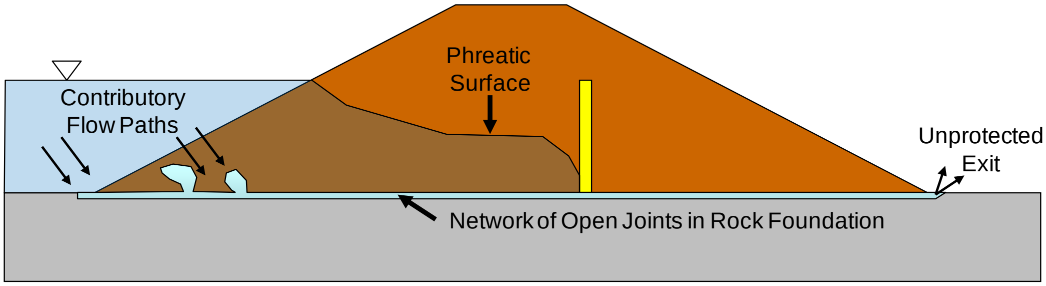


Figure D-6-3b Internal Erosion along the Embankment-Foundation Contact

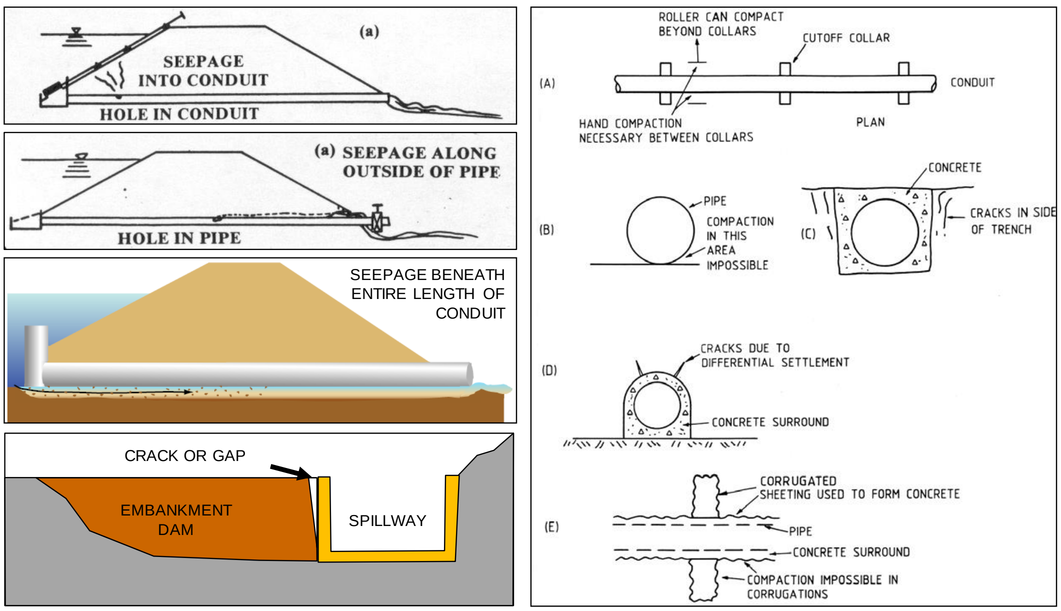


Figure D-6-4 Internal Erosion along or into Embedded Structures (adapted from FEMA 2005, 2008 and Fell et al. 2008)

It is important to note that no dam failures have occurred as a result of internal erosion into drains. This is most likely because this potential failure mode would take a long time to develop, and case histories indicate intervention through early detection has been successful in stopping the internal erosion process. The stilling basin case history described at the end of this chapter is an example of internal erosion into drains.

The locations of internal erosion identified here are <u>not</u> potential failure mode descriptions. The potential failure mode should be identified based on site-specific information and clearly described in detail from initiation to breach. It is important to identify where the failure path will likely form, where erosion first initiates, where the soil particles will be carried, how the erosion will progress, opportunities for detection and intervention, and how the embankment will breach.

D-6.4 Processes of Internal Erosion

Whereas the previous discussion centered on the locations of internal erosion cases, the following discussion describes the specific internal erosion "mechanisms" and processes that have been observed in case histories. The term "process" describes more phases of internal erosion than just the initiating mechanisms.

Organizations that have studied internal erosion incidents have observed several different mechanical processes and have classified those incidents by various mechanism or processes to establish some degree of common terminology along with an understanding of the physical factors associated with each type of internal erosion. Those evaluating internal erosion should consider the specific mechanics of the potential failure modes envisioned at a specific site and provide a full description of the entire process, regardless of the mechanism name. The general processes to consider are:

- Scour (concentrated leak erosion and soil contact erosion)
- Backward erosion piping
- Internal migration (stoping)
- Internal instability (suffusion and suffusion)

Flaws and other physical factors that can lead to an internal erosion failure mode and guidance for evaluating the probability of initiation of internal erosion are discussed in this chapter. Each of these processes are described below in detail.

D-6.4.1 Scour

"Subsurface erosion initiated by scour" was used by Terzaghi, Peck and Mesri (1996) in the latest edition of "Soil Mechanics in Engineering Practice" to describe the internal erosion process that occurred at Teton Dam. Scour is most likely to occur at or near the contact of an embankment with a jointed rock foundation, in cracks/defects in embankment fill, along conduits, and through transverse cracks near the top of an embankment.

This mechanism occurs when tractive seepage forces along a surface (e.g., a crack within the soil, adjacent to a wall or conduit, along the embankment-rock foundation contact) are sufficient to move soil particles into an unprotected area or at the interface of a coarse and fine layer in the embankment or foundation. This mechanism does not necessarily imply a backward (toward impounded water) development of an erosion pathway. Enlargement of an existing defect may occur anywhere along the seepage pathway and will occur first where the combination of hydraulic shear stresses and erodibility are most adverse.

Two subsets of this category include concentrated leak erosion and soil contact erosion.

D-6.4.1.1 Concentrated Leak Erosion

Sherard (1973) used the term "concentrated leak" to describe the flow out of cracks that extended through an embankment to distinguish this from seepage that flows through the pores of intact soil. Where there is an opening through which concentrated leakage occurs, the walls of the opening may be eroded by the leaking water as shown in Figure D-6-5. Examples of such concentrated leaks include through a crack caused by settlement or hydraulic fracture (Figure D-6-6) in a cohesive clay core, desiccation and tension cracks at higher levels in the embankment, cracks resulting from differential settlement of the embankment, or through bedrock discontinuities that erode adjacent embankment materials. Many of the situations in which concentrated leaks may occur are depicted in figures provided in Appendix D-6-C. In some circumstances, these openings may be sustained by the presence of structural elements (e.g., spillways or conduits) or by the presence of cohesive materials able to "hold a roof" below which an opening is sustained and the periphery of which is eroded. It may also occur in a continuous zone containing coarse and/or poorly compacted materials which form a system of interconnected voids. The concentration of flow causes erosion (i.e., scour) of the walls of the crack or interconnected voids.

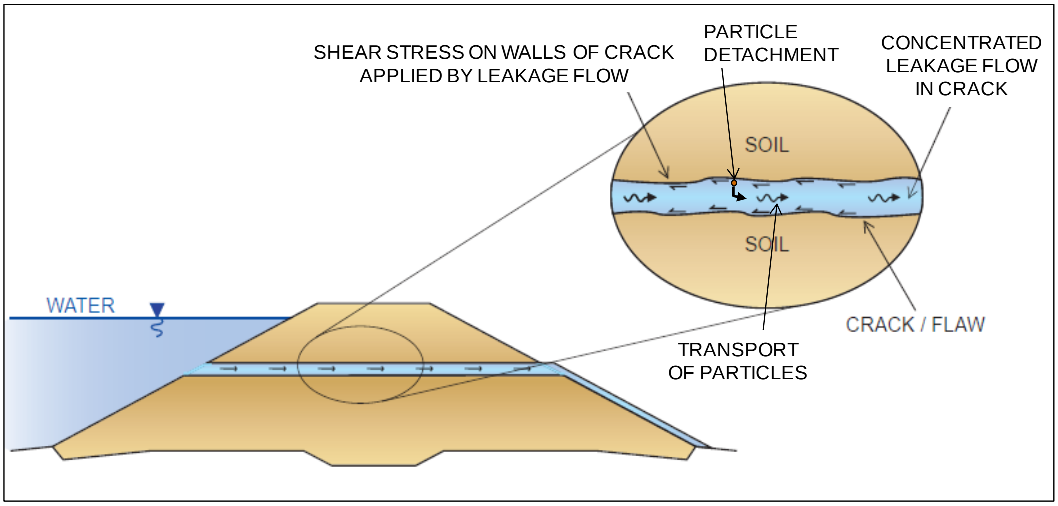


Figure D-6-5 Concentrated Leak Erosion(Courtesy of Mark Foster)

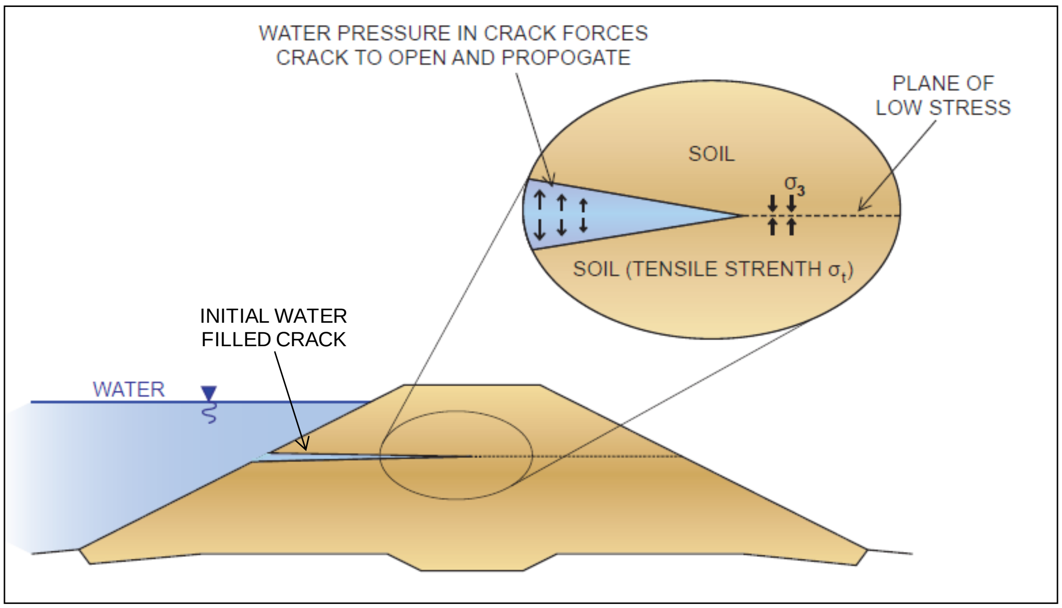


Figure D-6-6 Hydraulic Fracture (Courtesy of Mark Foster)

D-6.4.1.2 Soil Contact Erosion

Soil contact erosion describes the process in which a fine-grained soil is in contact with an open-work gravel, and flow parallel to the contact erodes the fine soil into and through the gravel. It is similar to concentrated leak erosion through a crack, but the flow is occurring through a gravel which is scouring fine materials in contact with the gravel. It is considered separately because there are specific methods that can be used to assess the likelihood based on: 1) the use of filter criteria to determine if erosion of the fines into the gravel is possible (i.e., not filtered); and 2) estimate if predicted velocities are high enough in the gravel to detach and transport fine particles. These methods differ from those used to assess scour in a crack mainly because the hydraulics of the flow are more complex in a gravel layer. The field conditions necessary for this process to occur are not common. The cases that led to the laboratory testing of this process were apparently a result of silt fill placed in contact with open-work gravel resulting in sinkholes or subsidence in some levees. The situation where the fine-grained soil is located over the gravel has been found to be much worse than the inverse as shown in Figure D-6-7. If soil contact erosion initiates, it could lead to one of the other processes identified in this chapter such as internal migration. It can lead to the formation of a roof at the interface, sinkhole development, creation of a weaker zone leading to slope instability, or clogging of permeable layers and increase in pore water pressure. Figure D-6-8 depicts some possible locations that contact erosion could develop.

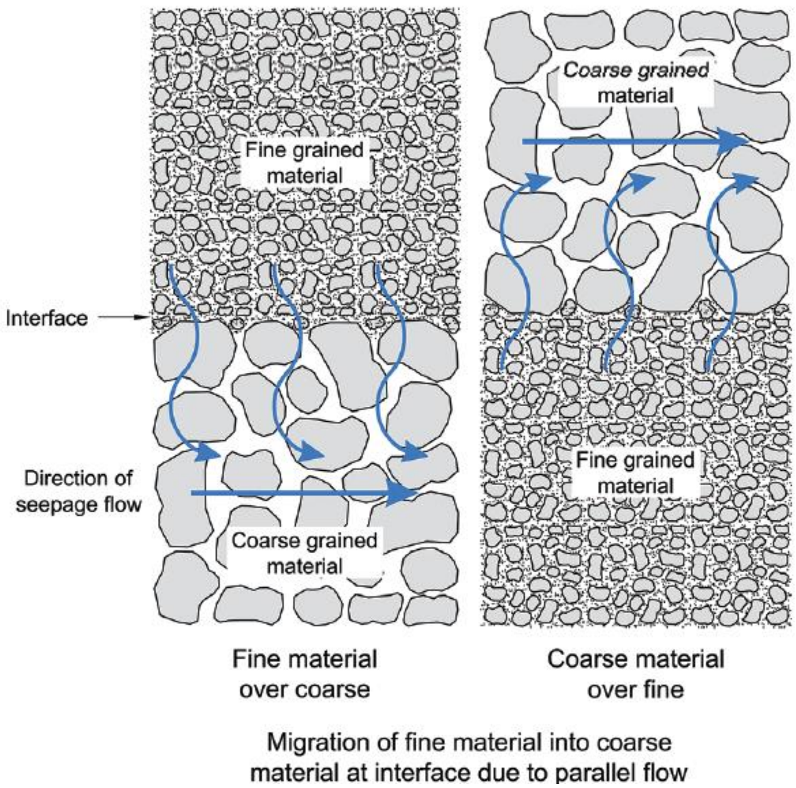


Figure D-6-7 Contact Erosion Process (International Levee Handbook 2013)

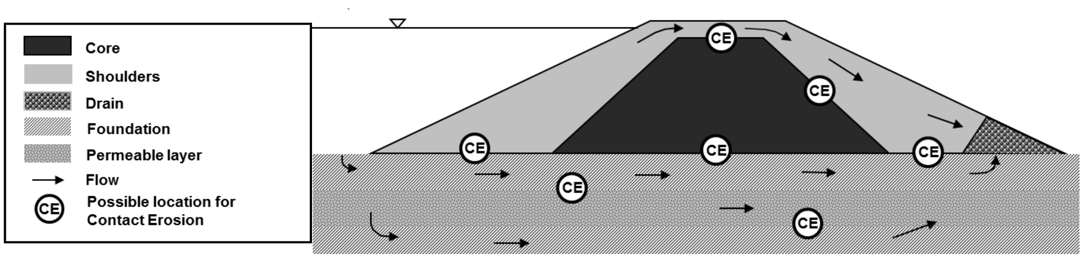


Figure D-6-8 Possible Locations of Initiation of Contact Erosion (Béguin et al. 2009)

D-6.4.2 Backward Erosion Piping

Backward erosion piping (BEP) occurs when soil erosion (particle detachment) begins at a seepage exit point and erodes backwards (towards the impounded water), supporting a "pipe" or "roof" along the way. As the erosion continues, the seepage path gets shorter, and flow concentrates in plan view, leading to higher gradients, more flow, and higher potential for erosion to continue. Four conditions must exist for BEP to occur: 1) flow path or source of water;

2) unprotected or unfiltered exit; 3) erodible material within the flow path; and 4) continuous stable roof forms allowing a pipe to form. BEP is particularly dangerous because it involves progression of a subsurface pipe towards the impounded water until a continuous pipe is formed, as shown in Figure D-6-9.

Backward erosion piping occurs in cohesionless soils or those with a low plasticity index (PI). It mainly occurs in foundations due to the potential for continuous low density deposits and gradients sufficient to initiate particle movement but may occur within embankments. The erosion process begins at a free surface on the landside or downstream side of the embankment. For BEP in the foundation, the free surface may be the ground surface, a ditch at the embankment toe, the stream bed further downstream of the embankment, or a defect in a confining layer (e.g., due to desiccation cracking, uplift or blowout, animal burrows, excavation, or other penetrations). BEP is often manifested by the presence of sand boils. Seepage and sand boils can represent a wide spectrum of potential conditions and risks (Von Thun 1996). Piping will develop when there is enough pressure and the supply of water from the pervious layer is sufficient. However, this implies the erosion will be slow when the pressure head which has caused the sand boil is sufficiently dissipated by the increased flow through the boil, similar to the effect of a relief well. For BEP in the embankment, the free surface may be an unfiltered or inadequately filtered zone downstream of the core.

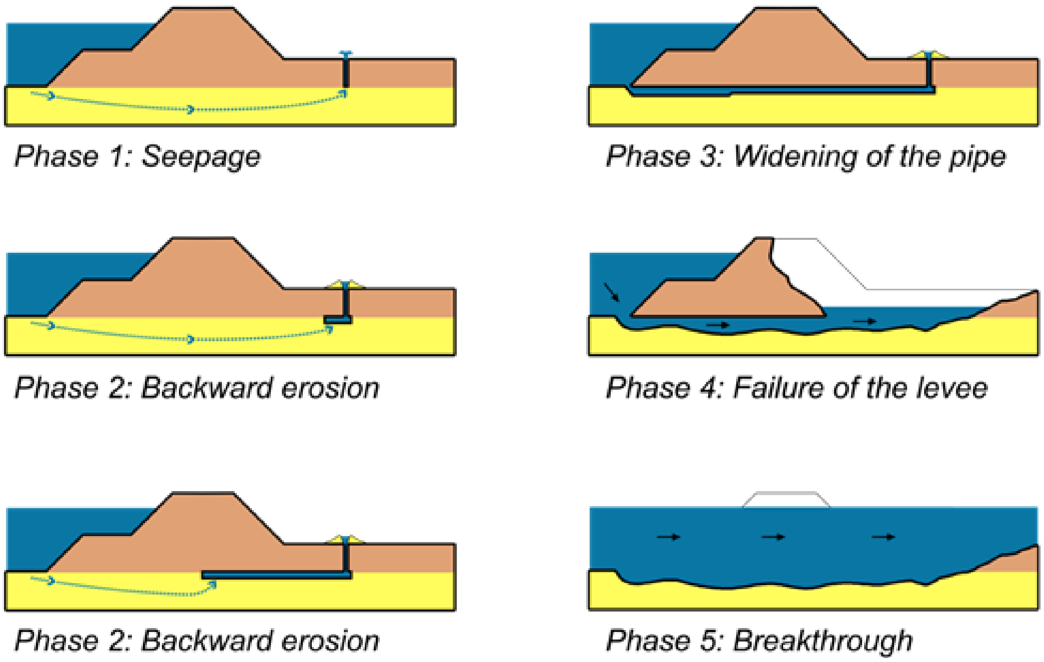


Figure D-6-9 Backward Erosion Piping (adapted from van Beek et al. 2011)

D-6.4.3 Internal Migration

Internal migration occurs when the soil is not capable of sustaining a roof or pipe. Soil particles move or drop into an unfiltered exit and a void grows until the temporary roof can no longer be supported. Soil particles that drop to the bottom of the void are carried away by seepage through the unfiltered exit. This mechanism is repeated progressively causing the void to enlarge and migrate vertically upward. These voids can develop in both saturated and unsaturated environments and typically result in formation of a sinkhole on the surface of the embankment. Soil particles migrate downward primarily due to gravity, but may be aggravated by seepage or precipitation, and a temporary void grows in the vicinity of the initiation location until a roof can no longer be supported; at which time the void collapses. This mechanism typically leads to a void that may stope to the surface as a sinkhole. Stoping can occur in narrow central core dams constructed with broadly graded cohesionless soils (e.g., glacial till) due to internal instability/suffusion, or in other embankments due to open defects in rock foundations or structures embedded in the embankment. This mechanism may be repeated progressively until the core is breached or the downstream or landside slope of a levee is over-steepened to the point of instability.

This could be a slow-developing potential failure mode, and successful intervention is likely if the void manifests above the water surface or the downstream or landside slope. The most critical location for a void to form is beneath the impounded water, as this opens up the potential to introduce full hydraulic head to a more downstream location. Figure D-6-10 depicts the potential locations over an inadequately filtered exit where internal migration could initiate.

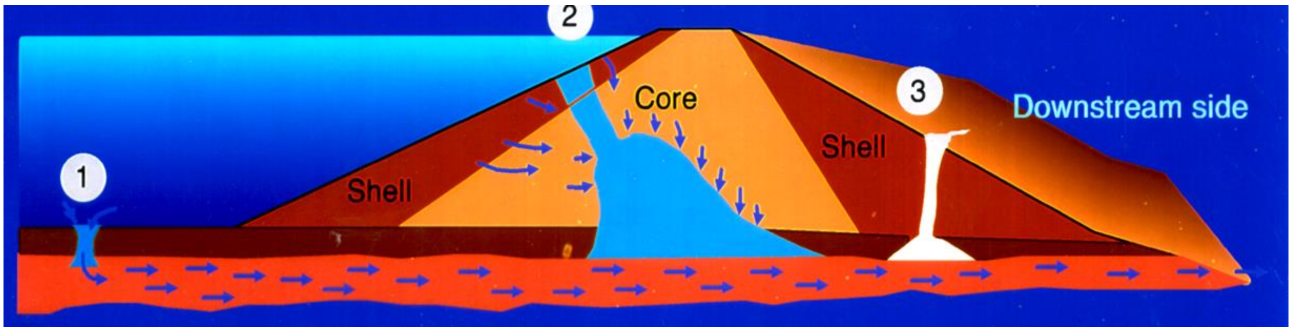


Figure D-6-10 Internal Migration

ICOLD (2015) describes a stoping process as global backward erosion and considers this a subset of backward erosion piping.

D-6.4.4 Internal instability- Suffusion and Suffosion

Suffusion and suffosion are both internal erosion mechanisms that can occur with internally unstable soils. It is more likely for these mechanisms to occur in complex glacial environments where tills, glacio-lacustrine, and outwash deposits co-exist, or in embankment zones constructed of these materials.

Suffusion is a form of internal erosion which involves selective erosion of finer particles from the matrix of coarser particles of an internally unstable soil, in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles, as shown in Figure D-6-11. Suffusion results in an increase in permeability (greater seepage velocities and potentially higher hydraulic gradients) and possibly initiation of other internal erosion mechanisms into/along the remnant coarser soil skeleton.

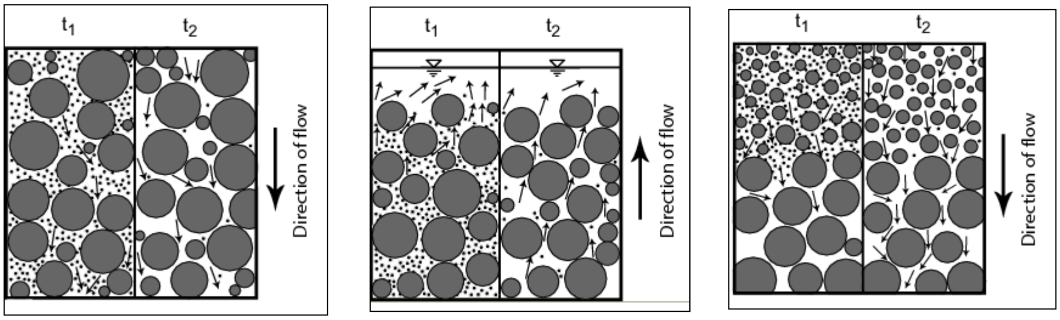


Figure D-6-11 Internal Instability (Suffusion) (adapted from Ziems 1969)

Suffosion is a similar process but results in volume change (voids leading to sinkholes or deformation of overlying embankment materials) because the coarser particles are not in point-to-point contact. Suffosion is less likely under the stress conditions and gradients typically found in embankment dams. This condition might require consideration of BEP, cracking and concentrated leak erosion, or soil contact erosion.

D-6.5 Conceptual Framework for the Internal Erosion Process

D-6.5.1 Internal Erosion Process

The process of internal erosion has been generally broken into four phases: 1) initiation of erosion (particle detachment); 2) continuation of erosion (inadequate particle retention); 3) progression of erosion (continuous particle transport and enlargement of erosion pathway); and 4) initiation of a breach. As an example, the first 3 phases of a case of scour initiating a failure mode through a zoned embankment are illustrated in Figure D-6-12.

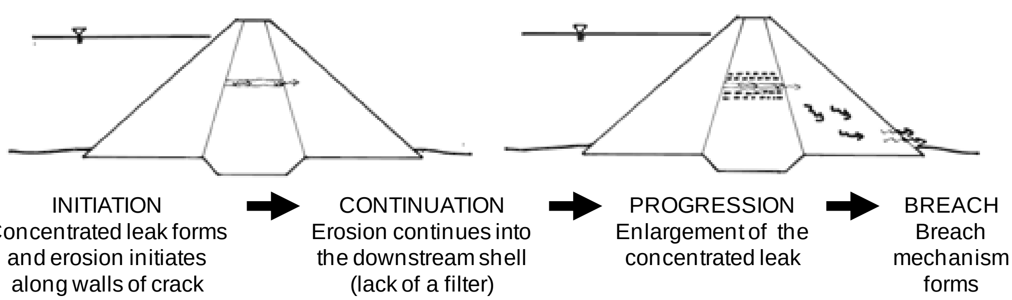


Figure D-6-12 Internal Erosion through the Embankment Initiated by a Concentrated Leak (adapted from Fell et al. 2008)

D-6.5.2 Event Trees

A generic sequence of events has been developed for analyzing internal erosion failure modes that is based on the four phases of internal erosion. In addition, a threshold water surface elevation (or several different ranges of elevations) and the likelihood of unsuccessful detection and/or intervention are assessed. These sequences of events can be illustrated as an event tree as follows: (Consequences are also evaluated for each event tree as discussed in Chapter C-1.)

⇳Water level at or above threshold level

⇳Initiation – Erosion starts

⇳Continuation – Unfiltered or inadequately filtered exit exists

⇳Progression – Continuous stable roof and/or sidewalls

⇳Progression – Constriction or upstream zone fails to limit flows

⇳Progression – No self-healing by upstream zone

⇳Unsuccessful detection and intervention

⇳ Breach (uncontrolled release of impounded water)

This event tree is applicable to scour related erosion through a zoned embankment. For other types of internal erosion processes, not all events may apply depending on the postulated failure progression and other site-specific factors. In addition, depending on how the potential failure mode is envisioned and on the information available, it might be appropriate to decompose the initiation event into two events: 1) flaw exists; and 2) erosion initiates given the flaw exists.

↳Water level loading (at or above threshold level)

↳Flaw exists – Continuous crack, high permeability zone, zones subject to hydraulic fracture, etc.

↳Initiation – Erosion starts

↳See above event tree for other events that apply.

The two-event approach is typically used for projects designed to include flood risk management and which have not been fully loaded. This allows the identification of scenarios where the likelihood of a flaw may be a primary factor in the risk estimate. The quantification of these events can provide a better understanding of how a flaw impacts both the estimate and the uncertainty in the risk estimate. When using historical base rates to estimate initiation (discussed in Appendix D-6-B), the one-event approach is typically used, as historical rates of a flaw existing where erosion has not initiated are unknown.

More details are provided in sections later in this chapter, as well as the detailed tables listing numerous factors to consider for each event included in Appendix D-6-J. Sample event trees for concentrated leak erosion and backward erosion piping are provided in the following figures. These examples illustrate the two-event approach. If the one-event approach is used, the first two events (flaw and initiation) would be replaced by a single event representing initiation.

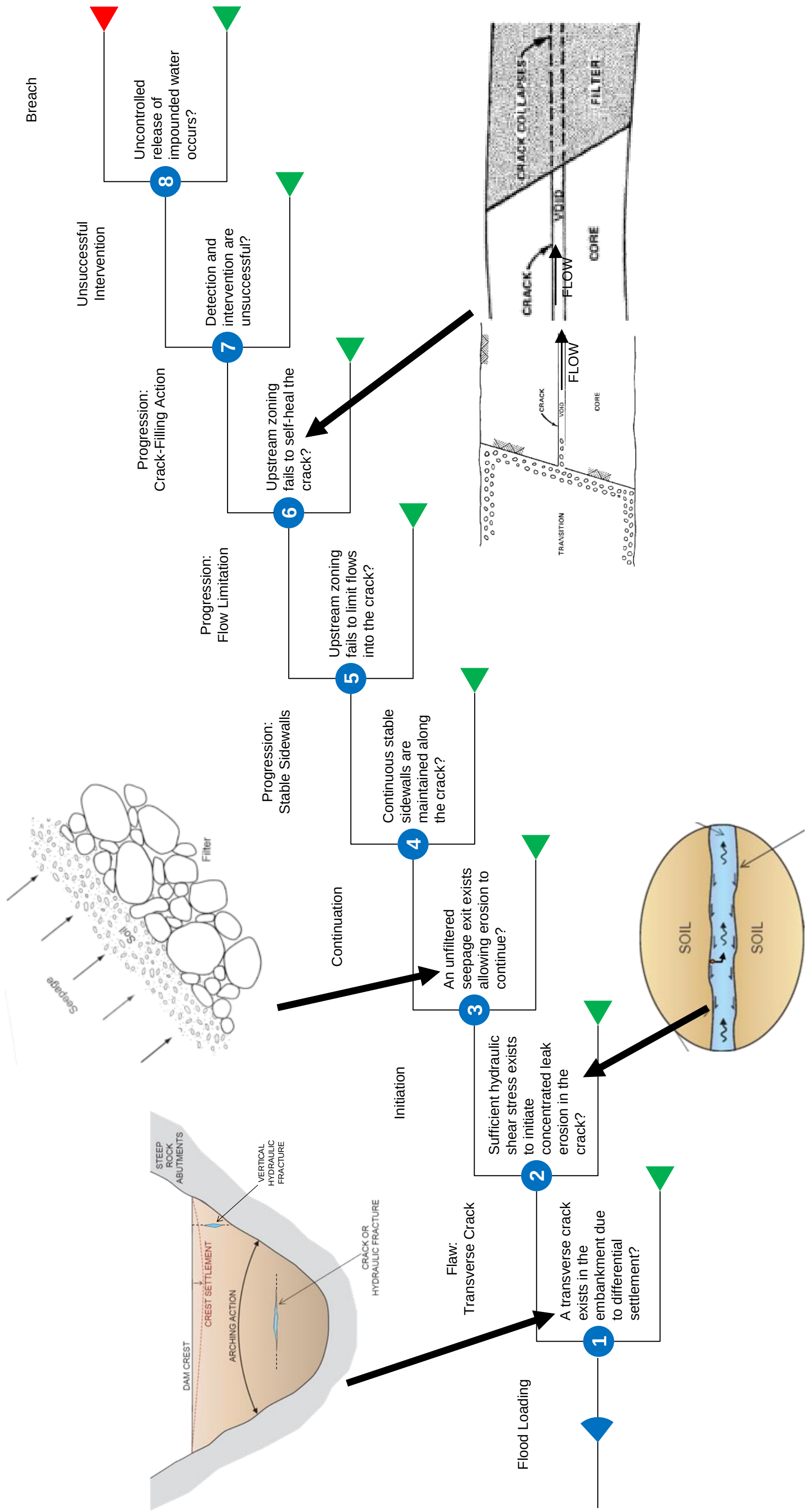


Figure D-6-13 Example of Event Tree for Concentrated Leak Erosion

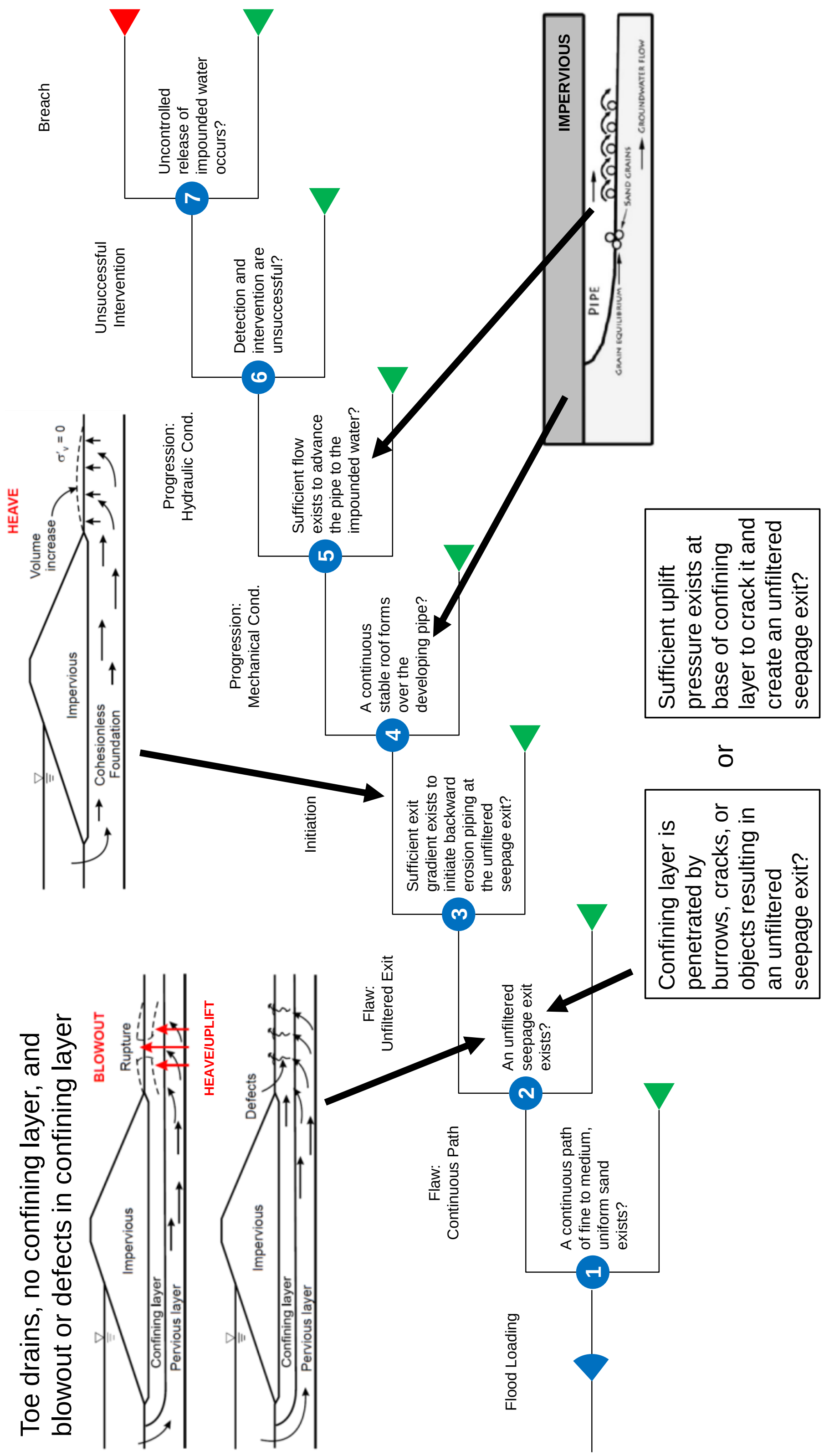


Figure D-6-14 Example of Event Tree for Backward Erosion Piping

The risk team should develop specific event trees for their identified potential failure modes. Sketches of the events on actual cross sections of the embankment and foundation can aid a team in conceptualizing each event in a potential failure mode and convey the information to decision makers.

D-6.5.3 Physical Locations of Failure Paths and Processes Combined

Based on all available information on the dam, the risk team will combine the physical locations of potential failure paths along with the potential process or mechanism likely at these locations. In many cases it will be a potential condition associated with natural soil deposits or bedrock upon which the embankment is founded or abutted, and in some cases it will be a potential condition in the embankment such as a crack or defect. The following is a short list of historic combinations along with some case histories that may be helpful to consider (every condition is not listed):

- Through the embankment –These have typically been attributed to flaws, defects or cracks that were identified or suspected to be caused by embankment conditions, foundation conditions or both. About 7 percent of Reclamation's dam incidents occurred at this location.
    - Caused by embankment conditions
        - Cross section details (Scour, BEP, IM) – Avalon Dam in New Mexico is possibly a case of a severe incompatibility between earthfill and rockfill leading backward erosion piping. Details are provided at the end of the chapter.
        - Construction related (Scour, II) – Poor compaction can lead to excessive settlement and cracking. Segregation can occur as the result of construction practices. Poorly treated layer associated with shutdowns of the fill operations can also lead to defects.
        - Rodent holes, tree roots, (BEP, Scour, IM) – A canal failure in Nevada was possibly a result of rodent holes as discovered in sections still remaining. Scour could have initiated in holes fully penetrating or partially penetrating holes could have resulted in backwards erosion piping.

        - Desiccation or freeze-thaw cracks (Scour) – Cracking or high permeability layers can occur near the crest if of susceptible soils are inadequate protected.

        - Internally unstable core materials (II, IM) Finer fraction may erode from these soils leading to sinkholes (Sherard 1979). WAC Bennet Dam is an example.

    - Caused by foundation conditions

        - Differential settlement of soil foundation (Scour) – Wister Dam in Oklahoma was a severe case of differential settlement likely causing embankment cracks that skewed across the dam aligned with soils left in the foundation resulting in concentrated leaks and scour. Details about incident are provided later in the chapter.

        - Near vertical abutments in rock (Scour) - Scoured materials could be carried both into joints in the foundation and or into an unfiltered zone in the dam as was likely the case at East Branch Dam in Pennsylvania. In this case, the core was likely cracked due to a near vertical step in abutment, the core was definitely scoured and much of the core material was transported by leakage to the toe through an unfiltered rockfill drainage layer in the embankment. Broadhead Dam also in Pennsylvania was another case where it appeared embankment cracks led to scour into a gravel drain that did not successfully filter. At the contact between the dam and foundation: The contact between the embankment and a rock foundation is a key place to evaluate. There are a number of case histories of this location, a couple of which led to failure. About 6% of Reclamation's dam incidents are likely to have occurred at this location.

    - Rock foundations

        - Continuous, open joints or bedding planes (Scour) - Scour of core materials can occur along pathways through an embankment but can also transport soils from the core into openings in bedrock such as likely occurred at the Fontenelle Dam in Wyoming, Teton Dam in Idaho, and Quail Creek Dike in Utah. See later in the chapter for details about the first two of these cases.

        - Open untreated joints beneath the shells of dams has led to internal migration and sinkholes vertically above the location of the open joints such as Churchill Falls Dykes, Canada (Sherard 1979).

        - Karst (IM, Scour) – The apparent collapse of karst features has led to collapse of overlying foundation soils due to internal migration at Wolf Creek Dam. At Clearwater Dam the collapse led to internal migration within the upstream shell and a sinkhole.

    - Soil foundation

        - Silt placed against gravel in foundation (SCE, IM). ICOLD Bulletin 164 Volume 2 cites a case history on the River Rhone in France.

- Through the foundation: This location was responsible for approximately 70% of Reclamation's dam incidents. This is the most common internal erosion failure mode for levees.

    - Soil Foundations

        - Natural impervious blanket over continuous, uniform, fine to medium sands (BEP) - A.V Watkins Dam in Utah was a case where the overlying blanket contained thin hard pan layers that formed the roof for backward erosion piping of underlying fine sands into a downstream ditch. Details of the case are provided later in the chapter. Bois Brule and Kaskaskia Island are cases of levees that breached due to BEP during the 1993 Mississippi River flood.

        - Internally unstable soil deposit (II, IM) – These cases have frequently led to internal migration as overlying non-plastic soils tend to collapse down into voids caused by the removal of fines in cases of internal instability. For a zoned embankment with an internally unstable filter, after washout of the fines occurs due to suffusion, the filter gradation may become too coarse to provide adequate particle retention.

        - Desiccation cracks (Scour) – Poorly treated foundation subgrades.

        - Earth fissures due to subsidence (scour) - This can be caused by ground water extraction, oil extraction, or underground mining. This can also lead to defects in the embankment.

        - Differential settlement of foundation soils (Scour, IM) At Mississinewa Dam Indiana it appears that foundation soils were damaged due to scour or internal migration into underlying karst which led to localized crest settlements roughly over the features. Later investigations revealed the embankment was likely damaged as well.

- Rock foundations
    - Soil-filled joints (Scour) - highly dependent upon geology and continuity

- Into or Along a through-penetrating structure
  - Flaw in embedded pipe (BEP) - Caldwell Outlet Works at Deer Flat Dams was a case of backwards erosion piping of the foundation soils into cracks in the conduit. See later in the chapter for details about this case. About 5% of Reclamation's dam incidents associated with conduits.
  - Along an embedded wall (Scour)
  - Along an embedded pipe (Scour, BEP) – There are numerous cases of levee incidents or breaches associated with poor compaction and construction details around corrugated metal gravity interior drainage pipes.
  - Out of open defect in an embedded structure (Scour, BEP): Flow out of a pressurized conduit.

- Into a drain – About 11% of Reclamation's dam incidents likely occurred into drains.
  - Broken structural drain on soil foundation (BEP, IM) - Broken stilling basin drains at Davis Creek Dam Nebraska likely initiated backward erosion piping beneath the conduit that resulted in internal migration as a sinkhole formed adjacent the control house. See the Case History section in this chapter for details.
  - Damaged embankment toe drain (IM, BEP) – typically resulted in sinkholes (IM).

D-6.6 Loading Considerations

Internal erosion failure modes can develop in response to a loading applied to the embankment or its foundation. The loading is generally characterized as either:

- Static/normal operation (i.e., reservoir level at or above a threshold elevation that would cause initiation of internal erosion)
- Hydrologic (i.e., related to a flood or reservoir level higher than the normal operating reservoir level)

- Seismic (i.e., earthquake causes deformation and/or cracking that would cause initiation of internal erosion)

The likelihood of achieving a certain peak water level can be estimated by developing reservoir or river stage exceedance relationships based on historical operations and floods. Water levels where seepage flows/boils are observed or a potential structural/geologic defect are loaded are potential thresholds for internal erosion and should be included in the peak flood load ranges used in the event. The loading conditions for seismic potential failure modes are discussed in Chapter D-8.

D-6.6.1 Dams Operated for Storage

For dams that have been nearly fully loaded, (i.e., the design normal water surface) it is typical to separate potential failure modes under normal operating (static) conditions from hydrologic and seismic-related potential failure modes. For reservoirs that serve primarily as water storage, it is not unusual that they fill nearly every year, and in such cases a value of 1.0 for this loading event is frequently assigned. Where the reservoir does not typically fill, the likelihood of achieving a high or threshold reservoir is typically based on reservoir exceedance curves. If the static evaluation of the risk at a dam includes the use of historic rates of initiation based on internal erosion incidents, care must be taken in the selection of the loading interval to start the evaluation of the flood loading to avoid double-counting the load. For flood loadings (which are considered to be hydrologic failure modes), an estimate of the likelihood of reaching the historical high and higher elevations must be determined from flood frequency analysis and possibly flood routings (see Chapter B-1 on Hydrologic Hazard Analysis).

D-6.6.2 Dams and Levees Operated for Flood Risk Management

For dams and levees operated primarily for flood risk management, or have significant flood storage and in any case have not been significantly loaded or are not significantly loaded very often, the loading can vary significantly from year to year. Therefore, the full range of flood loading must be considered without evaluating static loading separately. The “static” loading in this case is essentially included in the hydrologic loading evaluation. Cumulative plots of annual probability of failure, average annual life loss, and average annual economic loss associated with “normal” operating ranges or floods of interest can be used to evaluate and portray risks for various levels of loading (e.g., for reservoir levels up to conservation pool) and help identify critical load ranges that may be contributing the most risk.

For each potential failure mode, the risk team can establish load increments for evaluation. These can be used in developing a system response curve that relates the conditional probability of failure to the water level for the full range of loading. Non-linear portions of the loading or system response can unknowingly lead to results that are controlled by less well-defined portions of curves. Therefore, the water levels must be carefully selected to define the shape of the system response curve, especially at elevations where significant changes in the probabilities may occur. In general, partitioning of the flood loading should consider the following elevations:

- Elevation of the peak annual pool or stage
- Elevation where the probability of initiation of erosion becomes non-zero (e.g., bottom of a crack, elevation of rock defect, etc.)
- Geological features which occur above a particular level in the foundation (e.g., highly permeable gravel layer)
- Elevations where there is a documented change in performance (e.g., boils, high piezometric levels, etc.)
- Topographic features (e.g., major changes in foundation profile)
- Elevations corresponding to changes in design (e.g., top of filter, top of impervious core, or top of downstream berms)
- Record pool or stage elevation. This is an important elevation because the embankment and its foundation have been tested up to this level.
- Uncontrolled spillway crest or key elevations associated with controlled spillway operations or design overflow weir sections.
- Probable Maximum Flood (PMF) elevation
- Elevation of the embankment crest

The water levels do not have to be consistent between failure modes or with the stage-frequency curve as long as the full range of loading is covered. Typically, 3 or 4 water levels are selected, but the actual number should be adequate to define the shape of the system response curve.

D-6.7 Initiation – Erosion Starts

"Initiation" is the first event of the conceptual model of an internal erosion failure. Arguably, this is the most difficult event to evaluate and estimate and also the most important (i.e., tends to have the most potential impact on the estimated annual probability of failure).

Garner and Fannin (2010) developed a Venn diagram as shown in Figure D-6-15 to illustrate that internal erosion generally initiates with the unfavorable coincidence of 1) material susceptibility (e.g., low plasticity clay susceptible to cracking); 2) stress conditions (e.g., foundation geometry and construction practices are conducive to development of low stress zones in the embankment); and 3) hydraulic load occur (e.g., water level rises to crack and flow velocity in the crack is sufficient to initiate concentrated leak erosion).

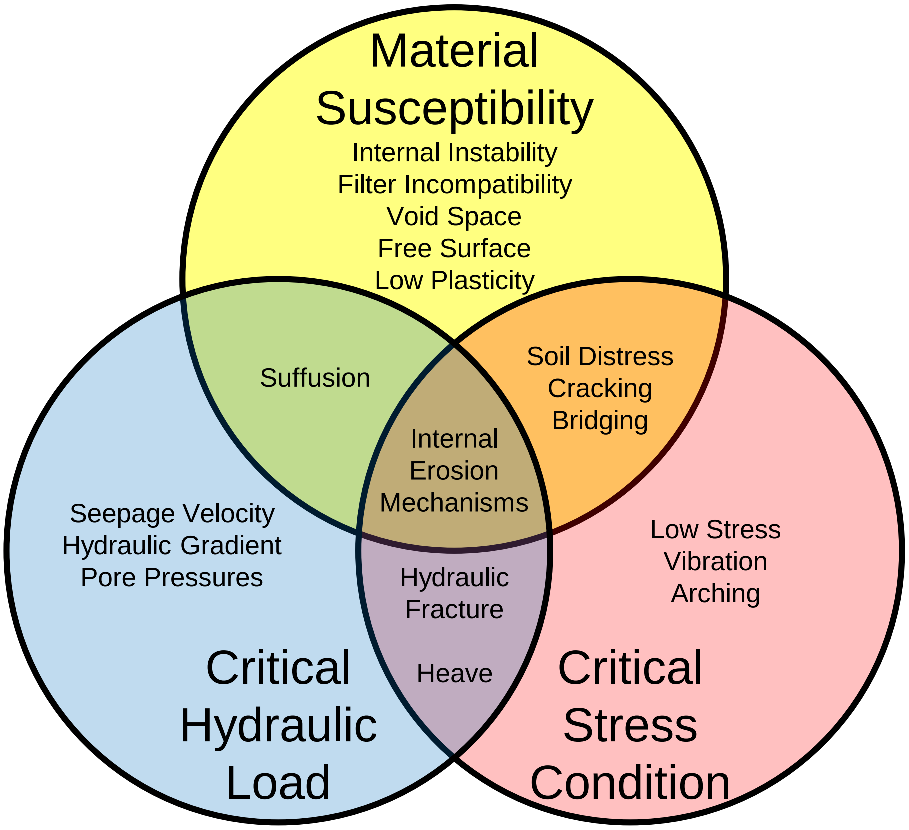


Figure D-6-15 Factors Affecting the Initiation of Internal Erosion (adapted from Garner and Fannin 2010 and ICOLD 2015)

The concept that the initiation of internal is in part dependent on material properties, hydraulic conditions, and in situ stress provides a good starting point to discuss each of these factors.

D-6.7.1 Effect of Material Properties on Initiation

D-6.7.1.1 Gradation, Particle Size, and Density

The actual in situ soil gradation and particle size as well as the continuity of a deposit or zone are important to all internal erosion failure modes. As particle size increases (as in coarser sands and gravels, cobbles, and boulders), it takes a higher seepage velocity (more energy) to move soil particles. It is important to recognize that laboratory gradations may not be representative of in-situ soils with larger particle sizes or soils susceptible to segregation or washout. Density of the soils plays an important role as well. The denser the soil, the harder it becomes to dislodge the soil particles and initiate erosion. Denser soil has lower permeability resulting in lower velocities of seepage, but this consideration will not apply to most, if not all, scour processes.

Density and plasticity also play a role in whether materials will be cracked or contain a flaw as discussed below.

D-6.7.1.1.1 Backward Erosion Piping

Laboratory testing by Schmertmann (2000) and various researchers in The Netherlands have shown that uniform soils provide significantly less resistance to backward erosion piping than broadly graded soils, while the latter may be susceptible to internal instability or suffusion as discussed below. This laboratory testing as well as forensic investigations of levee breaches in the United States generally indicate that backward erosion piping mostly occurs in the foundation, where the eroding soil is fine to medium sand with a coefficient of uniformity (C_u) less than about 3. Example uniform gradations susceptible to backward erosion piping are illustrated in the Figure D-6-16.

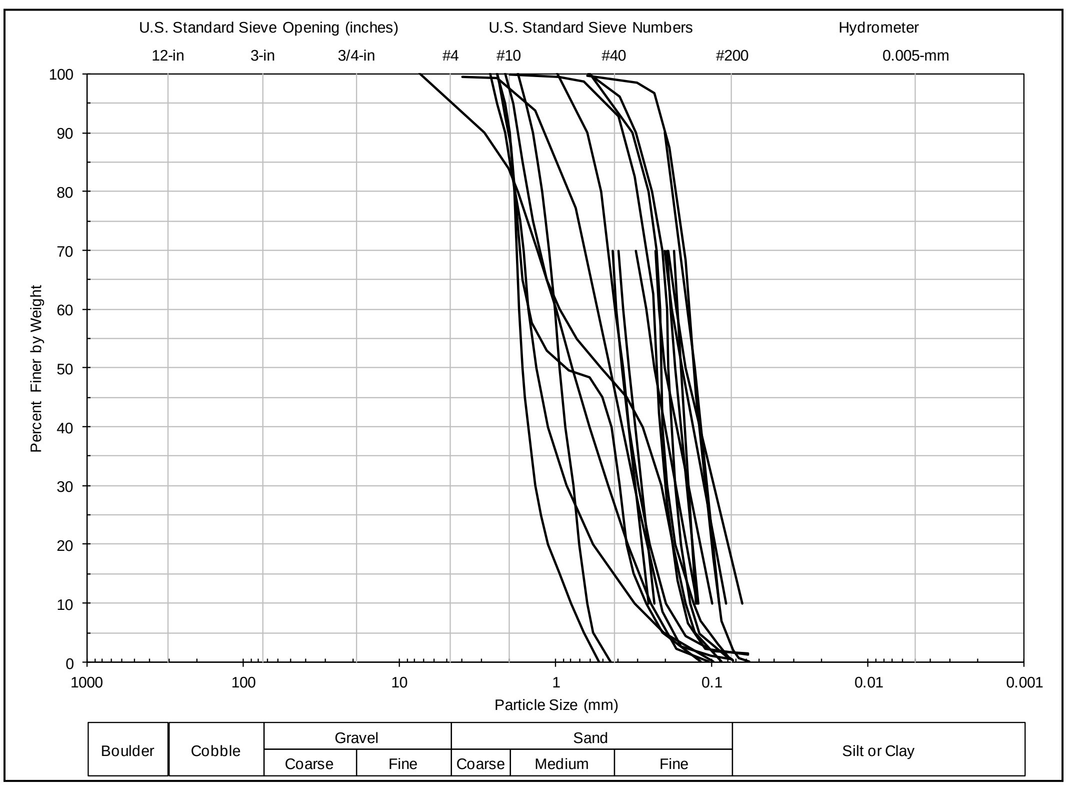


Figure D-6-16 Soils Susceptible to Backward Erosion Piping (de Wit et al. 1981, Townsend and Shiau 1986, van Beek et al. 2008, van Beek et al. 2009, van Beek et al. 2012)

D-6.7.1.1.2 Suffusion

Internal instability and suffusion are a concern for broadly-graded soils (i.e., soils with wide range of particle sizes – cobbles and gravels with sands, clays, and silts) with a flat tail of fines and gap-graded soils (i.e., missing mid-sized particles). Examples of these types of soils are shown on Figure D-6-17. Glacial soils can frequently fall into either of these categories.

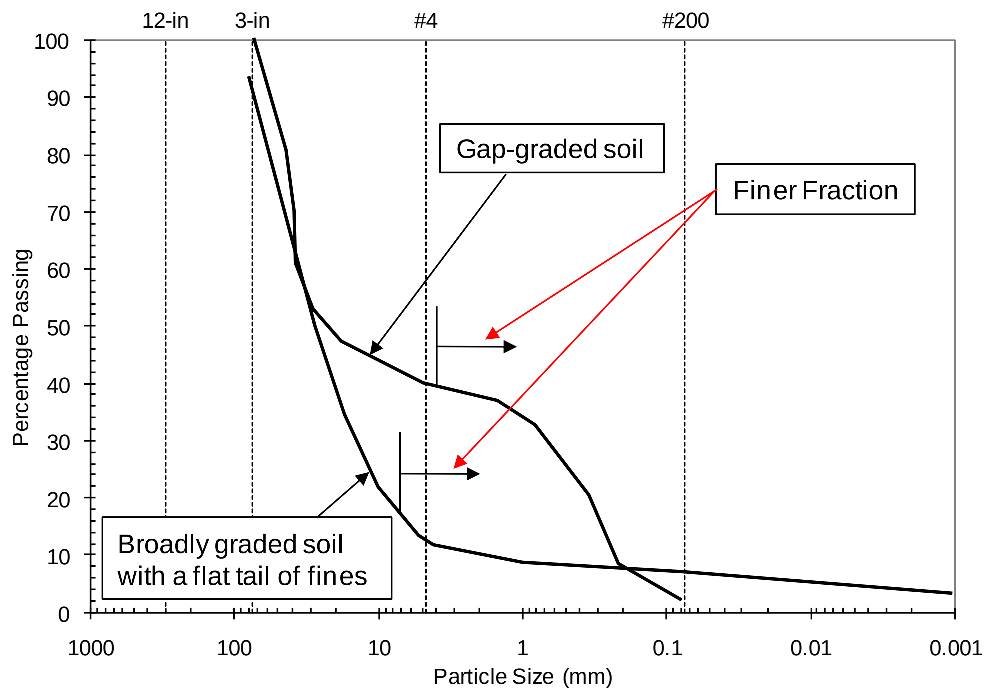


Figure D-6-17 Potentially Internally Unstable Soils (adapted from Wan and Fell 2004)

According to Sherard (1979), soils are generally considered "internally unstable" if the coarser fraction of the material does not filter the finer fraction. Sherard obtained data from a number of embankment dams, where sinkholes appeared on the crest and slopes of widely graded embankment embankments of glacial origin, and plotted a band around these gradations, as shown in Figure D-6-18. The internally unstable soil gradations usually plotted as nearly straight lines or as curves with only slight curvature within the range shown. Reclamation's filter design standard (USDOI 2011) also considers the slope of the gradation curve. This slope is illustrated in Figure D-6-18 and is noted as "4x." The slope of this line is approximately equal to the boundary slopes of Sherard's band. The location of the "4x" line on the plot is unimportant. Any portion of a gradation curve that has a flatter slope than this line indicates a potentially unstable soil, whereas portions of the gradation curve steeper than the line indicate a stable soil. This technique can also be used to evaluate gap-graded soils.

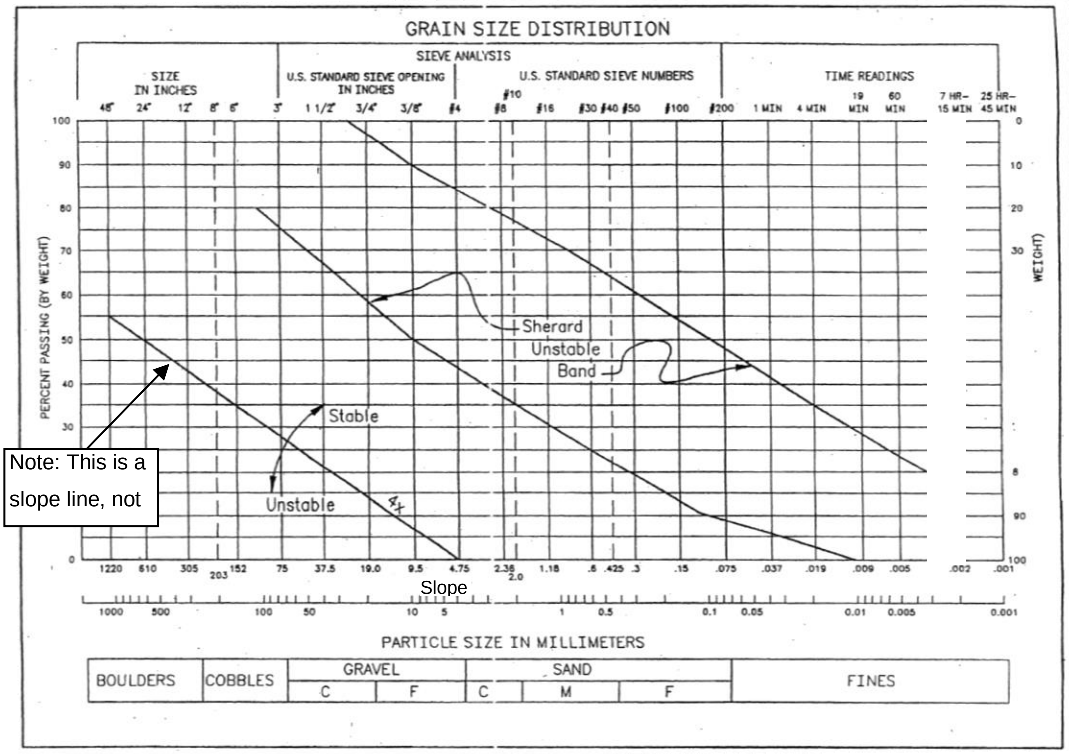


Figure D-6-18 Potentially Internally Unstable Soils (Sherard 1979 and Reclamation 2011)

D-6.7.1.2 Plasticity and Erodibility

Plasticity appears to be one of the most important factor affecting the potential for internal erosion to initiate. Based on an examination of Reclamation internal erosion incidents (Engemoen 2017), it is estimated that 80 percent of cases of all internal erosion at Reclamation embankments have been associated with soils of no to low plasticity (PI < 7). Backward erosion piping is simply far more likely to occur in cohesionless (or low plasticity) soils than in cohesive or plastic soils. Fell et al. (2008) indicate that the likelihood of backward erosion piping and suffusion in cohesive soils (PI > 7) is essentially zero under the seepage gradients which typically occur in embankments and their foundations. Plastic soils will also be less likely to experience other internal erosion mechanisms as well. This is apparent from case histories. The likely reason is that inter-particle bonding present in non-dispersive clayey soils provides additional resistance to leakage or seepage than in silts and coarse-grained soils.

Medium to high plasticity clay fines are less susceptible to cracking, hydraulic fracturing, or wetting-induced collapse settlement. However, the effect of plasticity varies with water content and compaction, with greater cracking potential in drier and stiffer/denser soils that would behave in a more brittle manner. Plasticity also affects the susceptibility of exposed embankment surfaces, including seasonal shutdown layers and staged construction surfaces, to environmental conditions. Medium to high plasticity soils are more susceptible to desiccation cracking than low plasticity to non-plastic soils, and low plasticity silts, silty sands, and silty gravelly and silty sandy soils are more susceptible to freezing than clean gravelly and sandy soils and high plasticity clays (Fell et al. 2008).

A key consideration in the likelihood of initiation of erosion is the erodibility of the embankment core and/or foundation materials. The likelihood of erosion initiating is much higher in highly erodible soils. Sherard (1953) published an early erosion resistance classification which is still useful in evaluating the likelihood of erosion, shown in Table D-6-1; the lower the number, the greater the erosion resistance. Note that plasticity and compaction moisture content plays a key role in erodibility. Table 8 of Sherard (1953) provides a more detailed examination of the soil characteristics from the case histories examined than Table D-6-1 for both backward erosion piping and cracking. Due to its size, that table is not reproduced in this document. ICOLD (2015) has prepared a similar classification for resistance to concentrated leak erosion based on Fell et al. (2008), as shown in Table D-6-2. Chapter D-1 describes the erodibility parameters including critical shear stress and erosion coefficient that are used in the prediction of erosion of soils subject to concentrated leak erosion.

Table D-6-1 Piping Resistance of Soils (adapted from Sherard 1953)

Greatest Piping Resistance Category (1)	1. Plastic clay, PI > 15, well compacted.
	2. Plastic clay, PI > 15, poorly compacted.
Intermediate Piping Resistance Category (2)	3. Well-graded material with clay binder, 6 < PI < 15, well compacted.
	4. Well-graded material with clay binder, 6 < PI < 15, poorly compacted.
	5. Well-graded, cohesionless material, PI < 6, Well compacted.
Least Piping Resistance Category (3)	6. Well-graded, cohesionless material, PI < 6, poorly compacted.
	7. Very uniform, fine cohesionless sand, PI < 6, well compacted.
	8. Very uniform, fine, cohesionless sand, PI < 6, poorly compacted.

Table D-6-2 Erosion Resistance of Soils from Concentrated Leaks (Scour) (adapted from ICOLD 2015)

1. Extremely erodible	All dispersive soils; Sherard pinhole classes D1 and D2; or Emerson Crumb Class 1 and 2. AND SM with FC < 30%
2. Highly erodible	SM with FC > 30%, ML, SC, and CL-ML
3. Moderately Erodible	CL, CL-CH, MH, and CH with LL < 65
4. Erosion resistant	CH with LL > 65

Dispersive soils are not addressed in Table D-6-1 but can be even more erodible. Dispersive soils are typically clays in which the clay particles can disperse or deflocculate (go into suspension) under still conditions, quite the opposite of most clays that require considerable seepage velocities to begin the erosion process. Dispersivity is related to clay mineralogy and particularly the electrochemical forces between soil particles as well as the pore water; soils having a high exchangeable sodium percentage are more susceptible. Dispersive clays are not limited to specific types, colors, geomorphology, or climatic conditions. Marine clays located in southern states where inland seas were present are often susceptible (e.g., Mississippi, Alabama, Arkansas, Louisiana, Texas, Oklahoma, New Mexico, and Arizona). It is difficult to tell whether a clay is dispersive without specific tests. However, it is common that some erosion features are observed in natural deposits of dispersive soils. Applicable laboratory tests that provide a measure of soil dispersivity include the (Emerson) Crumb test, the (Soil Conservation Service) double hydrometer test, the (Sherard) pinhole tests, and chemical tests that evaluate ESP (exchangeable sodium percentage) or SAR (sodium absorption ratio). It is frequently suggested that at least two different tests be run to check for dispersivity. Experience suggests initiation of internal erosion in dispersive clay has generally occurred on first reservoir filling or upon raising the reservoir to new levels for the first time (Sherard 1979).

D-6.7.2 Effect of Hydraulic Conditions on Initiation

D-6.7.2.1 Role of Concentrated Leakage or Seepage

Embankments and foundations are not completely impervious, and thus, virtually all dams and levees have some degree of seepage. It is not necessarily the amount of seepage that leads to internal erosion incidents; rather it tends to be whether concentrated leakage or seepage is occurring in soils that are susceptible to erosion and at sufficient velocities to detach and transport particles. In other words, the initiation of erosion typically requires a particular pathway that allows a concentrated flow within a generally limited or localized area or feature within an embankment or its foundation (e.g., cracks in the embankment, bedrock joints, open rock defects, erodible sand beneath a roof-forming material, etc.). General seepage models that portray seepage through porous media represented by large zones or layers feature an idealized situation that is generally unlikely to accurately portray the potential for internal erosion in most cases. Instead, it is the “weak link” or anomaly in an embankment or foundation where a concentrated flow is likely to occur and result in an incident. Such weak links or reasons for concentrated flows typically include the types of defects previously discussed, as well as naturally occurring pervious layers that are susceptible to erosion. Practitioners typically use available laboratory testing, research, and empirical evidence to probabilistically estimate internal erosion potential in risk analyses. References to consider in aiding these determinations are provided in the appendices.

D-6.7.2.2 Gradients

It is important to recognize that there are two types of gradients associated with seepage through porous soils and internal erosion: vertical and horizontal gradients. Vertical (upward) gradients are considerations in the potential for heave, uplift or blowout, and sand boils and can lead to unfiltered exits or potential initiating conditions for an internal erosion mechanism. Horizontal (internal) gradients through an embankment or its foundation play a key role in whether internal erosion will initiate and progress.

D-6.7.2.2.3 Vertical Gradients

Traditional soil mechanics or seepage discussions on critical vertical exit gradients (e.g., by Terzaghi and Peck and Cedergren) have typically only presented examples using sand foundations. The term "heave" was used to describe the condition when the saturated sand specimen, subjected to upward seepage flow in the laboratory, suddenly decreases in density and increases in permeability. This limit-state condition occurs when the seepage pressure on a plane in the specimen equals the weight of the specimen, and the effective pressure becomes zero. The traditional equation can be rearranged to solve for the upward hydraulic gradient (or critical vertical gradient) which is then further reduced to the more recognizable form in practice as the ratio of the buoyant unit weight of the soil (γ_b) to the unit weight of water (γ_w):

$$i_{cr} = \gamma_b / \gamma_w$$

This simplified relationship for the critical vertical exit gradient can also be expressed as the condition when the pore water pressure equals the submerged unit weight of the soil, and thus the effective stress is zero. At the critical vertical gradient in cohesionless foundations, a "quick" condition exists in the sand, and the foundation materials may "heave" or "boil" as shown in Figure D-6-19. Sand boils are an indicator of locations where the critical vertical exit gradient is close to or may have been reached.

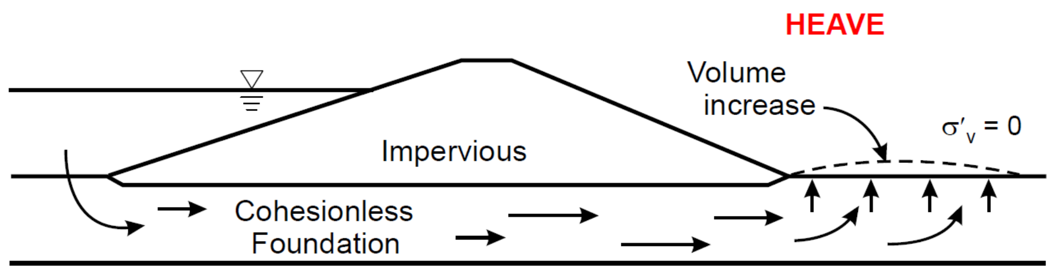


Figure D-6-19 Heave at the Toe of an Embankment (Pabst et al. 2012)

In the case of cohesionless foundations with no confining layer and vertical (upward) flow at the toe, vertical exit gradients (i_e) can be estimated using seepage analyses or piezometric data and then compared to the critical vertical gradient. Estimated vertical exit gradients less than the critical vertical gradient provide an indication that they may not be sufficient to create heave/boiling conditions at a seepage exit. Depending on the state of knowledge about given site conditions, there can be significant uncertainty with the estimated values of gradients. It should be noted that Darcy's flow equation is only valid until the critical gradient is reached. At the critical gradient, the sudden rearrangement of particles results in a sudden increase in discharge at the same gradient indicating the flow is no longer proportional to the gradient and permeability is no longer a constant. It is possible that sand boils may form, but significant particle transport may not occur due to other conditions, such as inability to hold a roof, heterogeneity of actual soil deposits, or insufficient horizontal gradients over a long enough time to fully develop an internal erosion mechanism.

A "blanket-aquifer" foundation consists of a low permeability, confining layer (such as clay) overlying a pervious layer (such as sand). If the pervious layer is not cut off upstream, seepage pressures in the pervious layer at the base of the confining layer may exceed the overburden pressure of the confining layer (i.e., soil blanket) at the downstream or landside toe of an embankment, and uplift (or "blowout") of the confining layer may occur as shown in Figure D-6-20. This is a primary concern for levees, and the term "heave" has also been used to describe uplift/blowout of the soil blanket by USACE and others in the literature. When soil blankets are ruptured, sand from an underlying aquifer will often be forced up through the confining layer, producing sand boils. A quick/boiling condition that often forms in cohesionless material may not exist around the sand boil, but "spongy" ground conditions are often noted and can be seen and felt when walking on a ruptured or an uplifted soil blanket.

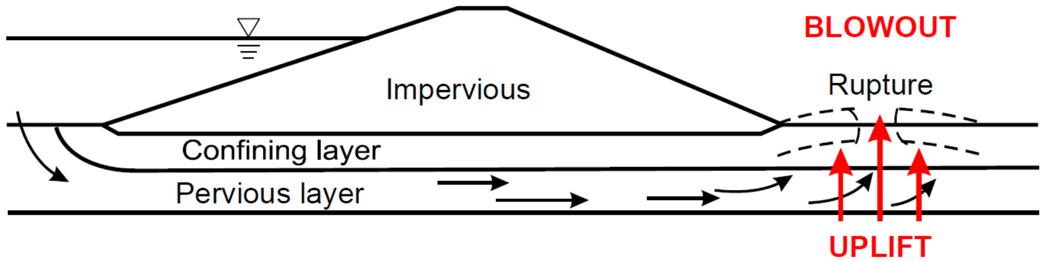


Figure D-6-20 Uplift and/or Blowout at the Toe of an Embankment (Pabst et al. 2012)

The limit-state condition for uplift of the confining layer is reached when the uplift pressure at the base of the confining layer equals the weight of the confining layer (at the time the corresponding uplift pressure is applied). If the uplift pressure in the field exceeds the weight of the confining layer at any time, uplift is likely to initiate and result in significant changes in the seepage regime. If uplift or blowout does occur, a potential unfiltered exit location is provided where internal erosion may initiate. As uplift occurs, a new seepage exit can form beneath the confining layer where hidden deterioration can occur from concentrated seepage if the horizontal gradients are high enough. With blowout, the confining layer is ruptured providing an unfiltered exit to the ground surface. The specific location of a rupture may be the result of a defect and/or the location of the maximum uplift pressure. If hidden deterioration was occurring before the blanket ruptured, these locations may be coincident. As previously mentioned, significant particle transport may not occur due to other conditions related to material erodibility, roof formation, and insufficient horizontal gradients to develop an internal erosion mechanism.

The assumptions for seepage conditions, tailwater conditions, degree of saturation, and density of the confining layer must be carefully considered in the evaluation of the limit state for uplift of the confining layer. Uplift can also initiate in partially saturated confining layers, especially for light weight soils (peat or OH soils) or cohesive soils in cases of drought.

For blanket-aquifer foundations, two methods have been used to evaluate uplift and/or blowout of the confining layer. One method involves simply comparing the uplift pressure acting at the base with the weight of the fine-grained soil blanket at the time the corresponding uplift pressure is applied. The other method is the critical gradient approach which is commonly used for cohesionless foundations with no confining layer and vertical (upward) flow at the toe. Both approaches are applicable to some embankments, for example embankments which have blankets that vary significantly in key properties such as erosion resistance (ML versus CH) or blankets that are discontinuous due to an old ox bow. When in doubt, both can be used to inform the risk team. The critical gradient method is used primarily for levees and involves comparing the actual gradient across the landside soil blanket (confining layer) to the maximum allowable gradient. An "underseepage factor of safety" is calculated assuming steady state-seepage conditions (USACE 2012), which reduces to the same form as the critical vertical gradient approach mentioned above. Text books and literature are not always clear in defining what approach is preferred, or even in distinguishing between the two approaches. When conducting a risk analysis, the team should consider the most appropriate methodology for their site-specific conditions to help them better understand the potential failure mode and its likelihood to develop. The potential for heave, uplift, and/or blowout in the field can is greatly influenced by geologic details, the details of man-made features, climatic conditions, as well as biological and chemical processes such as excavation by rodents, plugging of seepage exits by bio-fouling, or mineral deposition. Risk teams should be aware and consider these key factors and whether or not they are included in analyses.

D-6.7.2.2.4 Horizontal Gradients

Horizontal (or nearly so) gradients are internal gradients along a seepage or leakage pathway through an embankment and/or foundation. They affect the likelihood that internal erosion can occur by such means as concentrated leak erosion, backwards erosion piping, or suffusion. There is a fundamental difference between upward gradients and horizontal gradients. Upward gradients are resisted by gravity and relate to the potential for heave or uplift and the possible initiation of internal erosion. However, gravity is not a resisting force for a horizontal seepage exit such as in a ditch at the toe of the embankment, and little to no horizontal gradient is required for initiation of internal erosion.

A typical “critical” vertical (upward) exit gradient in cohesionless soils is often thought to be around 1.0 for a specific gravity of 2.7 (where heave is concerned) and higher for cohesive soils not subject to uplift. However, the magnitude of horizontal gradient that has led to internal erosion is much lower. For example, the horizontal gradient at Reclamation’s A.V. Watkins Dam incident was calculated to be 0.06, and the horizontal gradient at USACE’s Wister Dam, which suffered concentrated leak erosion, was reported to be 0.02 (but contained some dispersive clays). Horizontal gradients as low as 0.02 were estimated for levees along the Mississippi River in 1937, 1947, and 1950, as shown in Figure D-6-21. Evaluation methods for horizontal gradients are discussed in Appendix D-6-E.

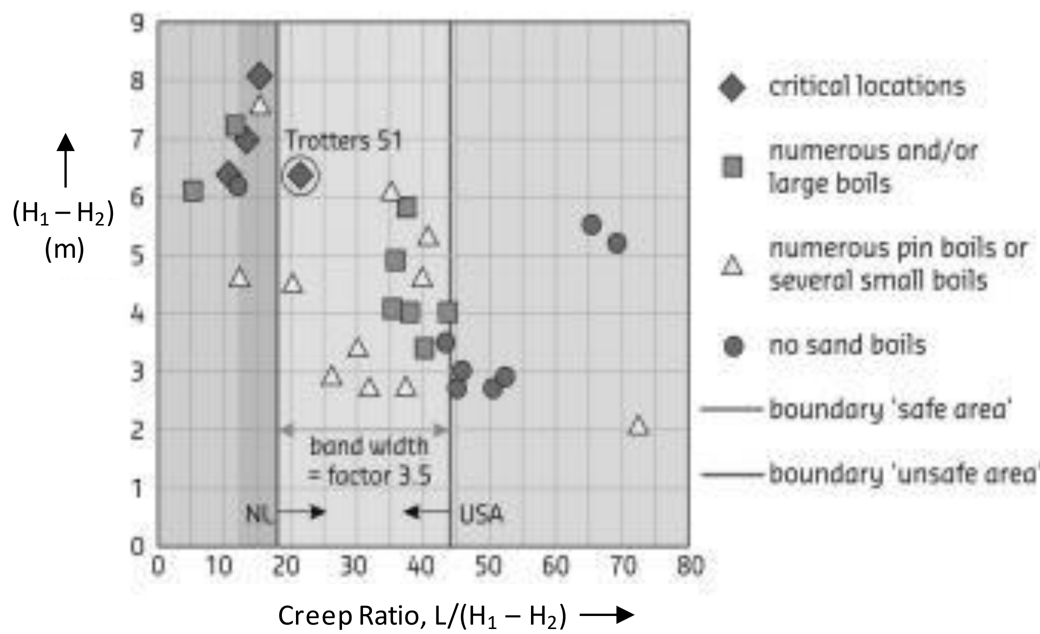


Figure D-6-21 Critical San Boil Locations along Mississippi River Levees (adapted from Ammerlaan 2007)

Unlike vertical exit gradients in sands, there is no widely accepted formula with which to evaluate when horizontal gradients might lead to internal erosion. In large part, this is due to no simple test to capture the physics that control whether or not it will occur. Unlike critical gradient piping by heave in cohesionless soils, no single laboratory test can be performed to determine the “gradient” at which erosion initiates and/or progresses. There is a great deal of uncertainty and variability inherent in lengthy seepage flow paths through embankment or foundation soils. These uncertainties include:

- Internal gradients are likely quite different at various places along the seepage pathway since natural, or even engineered, soils can be highly variable. The seepage path is undoubtedly not a straight line and likely meanders considerably, with seepage flows experiencing different amounts of head loss along the way.

- It is extremely unlikely that sufficient piezometers would be located in a number of critical locations along a seepage pathway in or beneath an embankment to accurately measure the piezometric pressures at key points in the critical (weak link) flow path.

- Furthermore, it is exceedingly difficult to accurately assess how the soils along an entire seepage pathway will respond to seepage gradients. Laboratory tests can provide insights into how a relatively small segment of representative soil will behave under various hydraulic gradients, and these studies suggest that key factors like soil plasticity and grain-size are important parameters in determining the potential for internal erosion. In actual field conditions, both soils and gradients are expected to vary in most instances.

Some studies indicate seepage forces on sand grains due to upward flow into the head of a developing pipe is a key reason that the critical horizontal gradient can be significantly lower than the critical vertical gradient. This is consideration is more significant when higher permeability layer exists beneath the eroding layer that helps concentrate flow into the developing pipe.

D-6.7.2.2.5 Concentrated leak erosion

Horizontal gradients can be used to estimate hydraulic shears stress for concentrated leak erosion, as described in Appendix D-6-C. For simplified crack or pipe geometries, the hydraulic gradient in this case is the hydraulic head difference divided by the length of the pipe or crack over which the hydraulic head difference occurs. As demonstrated by tests such as the Hole Erosion Test and Jet Erosion Test, cohesive soils are able to withstand much higher gradients than cohesionless soils before erosion initiates. However, this is not the case if the cohesive soils are dispersive. Case histories demonstrate embankments comprised of dispersive soils can erode quickly by internal erosion. As reported in Fell et al. (2008), highly erodible soils such as silts, silty sands, or dispersive clays may be likely to erode at a crack width of 0.25 to 0.5-inch under a hydraulic gradient as low as 0.1, and at widths as small as 1 or 2 mm under hydraulic gradients of 0.5 or more. Clays may not be likely to erode until cracks reach 1 or 2 inches in width and hydraulic gradients approach 0.5 or more. However, cracks in clays may swell shut upon wetting.

D-6.7.2.3 Velocity

Soil Contact Erosion and Suffusion: Seepage velocity can be used to as a measure of the hydraulic conditions for initiation of internal erosion. For soil contact erosion, the Darcy velocity is estimated and compared to a critical value to help assess the likelihood of initiation. See Appendix D-6-D for more details. In addition to hydraulic gradient and hydraulic shear stress methods, pore velocity can be used to assess the hydraulic conditions for suffusion. These methods are referenced in Appendix D-6-F

D-6.7.3 Effect of Stress Conditions and Presence of "Flaws or Defects" on Initiation

D-6.7.3.1 Low Stress Zones and "Arching"

The formation of low stress zones, or even tension zones, in an embankment is known to have led to many failures and incidents involving internal erosion. The zones can occur in areas of severe differential settlement. Foundation anomalies and conduits in narrow trenches have led to numerous instances of cracking and potentially hydraulic fracturing. In many cases, these low stress zones essentially lead to flaws or defects.

D-6.7.3.2 Conditions Associated with Initiation of Internal Erosion

This abbreviated list provides insights into what in-situ conditions increase the likelihood of an internal erosion process initiating. Appendix D-6-G contains a more detailed list.

Leads to increased likelihood of scour:

Through the embankment

Cracking from differential settlement of embankment materials due to shape of abutment or foundation or details of embankment cross section

Cracking of embankment due to differential settlement in foundation soils

Defects due to construction

Cracking due to exposure (desiccation or freeze-thaw action)

Animal burrows and vegetation

Cracking due to an earthquake

Through the foundation

Untreated soil-filled joints

Silt and fine sand deposits against open-work gravels

Earth fissures due to subsidence caused by ground water extraction, oil extraction, or underground mining(can also lead to scour through the embankment)

From the embankment into the foundation or along the embankment-foundation contact

Open joints, seams, faults, shears, bedding planes, solution features, or other discontinuities at contact with rock foundation.

Silt embankment constructed directly upon openwork gravel.

Along structures

Cracking of embankment associated with conduits

Poor compaction due to shape of structure

Leads to increased likelihood of internal migration:

Through or in the embankment

Broadly graded core placed in contact with coarse grained fill(typically happens along a steep core/shell contact)

Through the foundation

Fine grained soils over coarse grained open work soils (such as reservoir sediment over glacial outwash gravels)

From the embankment into the foundation or along the embankment-foundation contact

Embankment soils in contact with untreated open joints, seams, etc. in foundation rock

Non-cohesive or low PI embankment fill founded upon against openwork gravels.

Into structures

Damaged conduit

Into drains

Broken drain pipes or inadequately designed drain system

Leads to increased likelihood of backwards erosion piping

Through Embankment

Severe filter incompatibility at core contact with shell

Animal burrows,

Scour of impervious layer on riverside slope of sand levee

Through the foundation

Continuous uniform fine to medium sand underlying a roof-holding material

Thin natural impervious blanket overlying fine sands connected to river or reservoir

Penetrations or defects in the blanket

Riverside scour of the blanket

From the embankment into the foundation

Core material placed directly against open-work gravels in bottom of downstream side of cutoff trench

Along or into structures

Poor compaction around a structure

Damaged conduit surrounded with non-cohesive embankment or founded upon fine sands

Into drains

Broken drain pipes or inadequately designed drain system

Leads to increased likelihood of internal instability

Through the embankment

Non-plastic broadly graded thin core against coarse shell and defects due to construction (segregation)

Through the foundation

Broadly graded and gap-graded soil deposits such as some glacial deposits

D-6.7.3.3 Influence of Stress Conditions on Internal Instability

As described earlier, suffusion and suffosion deal with finer particles being washed out by seepage flows through a broadly graded or gap-graded, internally unstable soil. Stress conditions play a role in this process, specifically relating to whether the space between coarse particles in the soil is "over-filled" or "under-filled." When the coarser-grained portion of an internally unstable soil are essentially in point-to-point contact, the space between coarse particles can be thought of being under-filled, and the stresses are being carried by those point-to-point contacts of the coarse particles. Thus, the finer matrix material feels little to no stress and can consequently be washed out by seepage flows. However, when the larger particle sizes are essentially floating in the finer matrix material, there is no load carried by a coarser skeleton. Thus, all particles generally experience the same stress. In this scenario, the stress conditions upon all soil grains would require a much higher seepage velocity to move the materials. This is why suffusion (or the erosion of finer soils within an internally unstable soil with point-to-point contacts of the coarser grains) is much more likely to occur than suffosion (which features the coarser grains floating in finer matrix material) under the gradients typically present in embankment dams and levees and their foundations.

D-6.7.4 Approach to Estimating the Probability that Internal Erosion Will Initiate

Given an open or unfiltered exit exists, in most cases the key event in the event tree generally used to estimate the probability of an internal erosion failure is the probability that internal erosion will initiate. Initiation is typically judged to have a relatively low probability of occurring; is based on a number of variables including presence of a concentrated seepage path (flaw), length of the seepage path, hydraulic "gradients," and soil erodibility; and is thus difficult to estimate.

Developing the probability of initiation is typically completed using either an empirical approach or using analytical methods and application of researchers' findings on the potential for various soils to erode under various conditions. Neither approach should be considered more "right" as both approaches have merits. Furthermore, there may be situations when either (or both) approach might be more appropriate, and risk teams should not feel constrained to a particular approach. Regardless of the approach, the best practice is to utilize a process of evaluating all information and existing conditions at a site to flesh out potential failure modes and utilize expert elicitation to estimate the likelihood that the internal erosion process will initiate based upon the totality and strength of the evidence.

D-6.7.4.1 Empirical Approaches

Empirical approaches are based on the evaluation of internal erosion incidents or failures. The fundamental reason for an empirical approach is the difficulties and numerous uncertainties associated with applying laboratory findings and gradient assumptions to the spatially vast and variable embankment/foundation system typically being evaluated.

The statistics of historical failures and incidents can provide some insight when estimating the likelihood of initiation of internal erosion. However, such rates should be used with caution based on the general method in which they were developed and the inventory of dams and levees used to develop the rates. Whenever historical rates are provided, they may be more representative of the product of several probabilities on the internal erosion event tree. The rates must be carefully considered based on site-specific information. Seepage from the downstream or landside slope (from visual observation, measurement, or other non-invasive methods), measured high pore pressures, settlement, deformation, and cracking are possible indicators of a flaw or defect in the embankment. Similarly, seepage at the downstream or landside toe is a possible indicator of a flaw or defect in the foundation. The influence of observations on the probability of a flaw should take into account the mechanism causing the flaw, the available data, and the relative importance of the observations.

One empirical approach was developed by Fell, Wan, and Foster (2004) based on records of dam incidents (Foster et all 1998, 2000). The study provided an estimated probability of initiation of erosion for the categories of internal erosion through the embankment, associated with conduit through embankment, through foundations in soil or erodible rock and from the embankment into soil or rock foundation. Initiation estimates for first filling, reservoir above historic high and normal operation conditions were reported.

A second empirical approach was developed by Reclamation (Engemoen and Redlinger 2009, Engemoen, 2011 and 2017) based solely on the incidents in the Reclamation inventory. In general, Reclamation embankments were constructed with wide cores (long seepage path) often flanked by shells of sands/gravels/cobbles (providing some filtering capability) and good compaction techniques. Appendix D-6-B provided additional details on the use of Reclamation's historical frequencies.

D-6.7.4.2 Analytical Approaches

A number of methods, tests, and tools are available to assist in evaluating the probability of initiation of internal erosion. These in combination with observations and the experience of the risk team provide the evidence against which the probability estimates are made. The risk team discusses these and other factors that were identified and decides which should receive the most weight.

Details of some of the analytical methods and tests which the team would consider as a "more likely" or "less likely" factor during an elicitation for the probability of a flaw existing or probability of initiation are provided in the following Appendices:

- Appendix D-6-C: Concentrated Leak Erosion
- Appendix D-6-D: Soil Contact Erosion
- Appendix D-6-E: Critical Gradients for Evaluation of Backward Erosion Piping
- Appendix D-6-F: Internal Instability (Suffusion)
- Appendix D-6-G: Detailed List of Conditions Which Increase the Likelihood of Initiation of an Internal Erosion Process

D-6.8 Continuation

Erosion once initiated will continue unless the eroding forces are reduced or the passage of the eroded particles is impeded. Evaluation of this phase of internal erosion relies primarily on examining the filter compatibility of adjacent zones and layers in an embankment and foundation. In modern embankments, filters are used to prevent the migration of fines between various zones of the embankment or its foundation and to safely protect against leakage through cracks should they occur. Existing embankments without modern filters depend on the filter compatibility of core and transition/shell materials (if present), as well as the compatibility at the core/foundation contact.

In essence, continuation is the phase of internal erosion where the relationship of the particle-size distribution between the base (core) material and the filters or adjacent materials controls if erosion will continue. The methodology to evaluate the probability of continuation of internal erosion will vary depending on the seepage exit. Generally, three exit conditions are considered: 1) open exit; 2) filtered/unfiltered exit; or 3) constricted (non-erodible) exit. Chapter 5 of Reclamation's Design Standards No. 13 (Reclamation 2011) entitled "Protective Filters" provides guidance for design and construction of soil filters, drains, and zoning of embankment dams that is useful for consideration in risk assessments of existing dams. Modern filters and drains defend against cracks and assure significant head loss occurs at the boundary if it already has not occurred because of cracks. For a filter designed used current criteria and constructed using quality materials and good construction oversight and documentation, the probability that an unfiltered exit exists would typically be very low.

D-6.8.1 Continuity

An important point about continuation is whether the unfiltered exit is truly continuous. Zones in shell materials and layers of alluvial materials that act as unfiltered exits (i.e., don't satisfy filter compatibility) need to be continuous to an open face or extensive void space need to exist in coarse soils or bedrock for eroded fines to be deposited into.

D-6.8.2 Open Exit

If there is a free or open face, then there is no potential for filtering action due to an unfiltered exit, and the probability of continuing erosion is virtually certain (i.e., $P_{CE} \approx 0.999$). Open exits can also be the result from common-cause cracking in the filter or transition materials.

D-6.8.3 Filtered/Unfiltered Exit

Some zones may be designated as "filters" but may not satisfy the current definition of a filter. Conversely, there may be material that does not meet filter criteria but can be considered an "opportunistic filter." The evaluation of filter compatibility between the base soil (core) and the filter or adjacent materials requires representative particle-size distribution data. When there are a greater number of gradations, the reliability of the filter compatibility assessment is improved. If the gradations are plotted on the same sheet, the normal range or gradation band can be observed for both the base soil and the filter, along with any outlier gradations. In soils containing coarse particles (gravel, cobbles, or boulders), it is important to realize they frequently do not show up in gradations. In zoned embankments, multiple filters or zones often provide transition from the finer to coarser materials. Each filter zone must be filter-compatible with the preceding zone if seepage across the boundary occurs. Therefore, the filter compatibility evaluation may be a multi-step process, depending on the embankment zoning. If a perforated pipe is installed in drain rock to transmit accumulated water, the compatibility with the perforations in the drain pipe must be evaluated. The following steps should be followed when assessing the likelihood of continuation of internal erosion for filtered exits:

- Gather the available information on particle-size distributions of the core or foundation and the filter or transition materials. This may include data from design-phase borrow area investigations, construction control testing, and post-construction testing on samples from the dam or its foundation. If only a few samples are available for each zone and only from borrow sources, care must be taken in drawing conclusions from the data to evaluate filter compatibility. Consider reviewing the borrow source and placement information. It may be that different portions of the embankment were placed using different borrow areas or zones within a borrow area. Therefore, some areas may have predominantly finer core material (or coarser adjacent material) and these areas should be evaluated using information specific to those areas and not the average conditions. A check if scalping was done in the field using a grizzly (i.e., screen) can be important.

- Plot the particle-size distributions for the base soil and filter materials. If the base soil contains gravel (i.e., materials larger than a No. 4 sieve), then re-grade the base soil if any of the following conditions apply: the base soil contains greater than 15 percent fines; the base soil is gap-graded; or the base soil is broadly graded ($C_u \geq 6$ and C_z between 1 and 3). If none of these conditions apply, re-grading is not required. Regrading is performed to correct for potential internal instability and a large $D_{85}B$ that could result in too large of a $D_{15}F$ size to provide adequate filtering. Reclamation (2011) provides an example of re-grading calculations.

- Consider whether the filter materials are susceptible to cracking based on fines content, cementation, or the presence of plastic fines. If the filter materials are susceptible to cracking or subject to deformations that could cause cracking, then assume there is questionable potential for filtering action due to an unfiltered exit, and there is some probability (perhaps high) of continuing erosion (estimated by team judgment).

- Consider whether the filter materials are susceptible to segregation during storing, hauling, dumping, spreading, and compacting, and if the segregated layer is continuous. If a continuous segregated layer is likely, then the procedure of Fell et al. (2008) could be used to estimate the gradation after segregation.

- Consider whether the filter materials are susceptible to internal instability as described in Appendix D-6-F. If internal instability is likely, then the procedure of Fell et al. (2008) could be used to estimate the gradation after washout of the erodible soil fraction.

- Assess if the filter materials will prevent continuation of internal erosion using modern filter design criteria. For filter materials which are coarser than required by modern filter design criteria, the Foster and Fell (1999, 2001) method may be used because it allows assessment of filters which are too coarse to satisfy modern no erosion design criteria. The "No Erosion" criteria must always be used for the design of a new filter. The other criteria are only used to evaluate existing dams.

- Check for blowout in cases where there is limited depth of cover over the filter material, comparing the seepage head at the downstream face of the core to the weight of soil cover (see section entitled "Critical Gradient, Heave, Uplift, and Blowout"). In addition, check for possible slope instability assuming appropriate pore pressures.

Further details on these factors are provided in Appendix D-6-H.

D-6.8.4 Constricted (Non-Erodible) Exit

For erosion to continue, the open joint, defect, or crack in conduits, walls, or rock foundations needs to be sufficiently open to allow the surrounding soil particles to pass through it. The effective opening size of such defects can be used to assess whether such features will allow internal erosion to continue. Poorly designed or inadequately filtered underdrains, toe drains, relief wells, or weep holes into which embankment or foundation materials can be eroded should be evaluated using similar "opening size" considerations, where applicable.

There are no commonly adopted criteria for assessing the likelihood of continuation for this scenario, although some have used filter design criteria for perforation size for drain pipes. For example, in order to prevent erosion into a drain opening, the maximum pipe perforation dimension when designing drain pipes should be no larger than the finer side of the $D_{50}E$ of the surrounding envelope material. Since this criteria is used for design and could be assumed to represent no erosion limits, it is likely conservative unless the particles are flat.

Based on the results of filter tests on uniform base soils, Sherard et al. (1984) concluded that uniform filters act similar to laboratory sieves, with an opening sieve size approximately equal to $D_{15}F/9$. In a later series of tests (Sherard unpublished Memo 1G, 1985a), materials passing through the filters were caught and gradations of the material showed that approximately 97 to 99 percent of the particles were finer than $D_{15}F/9$. Foster and Fell (1999) obtained similar results. Based on these findings, Fell et al. (2008) suggested the following criterion for continuing erosion:

$$JOS_{CE} \geq D_{95}E \qquad \textbf{Equation D-6-1}$$

where JOS_{CE} = the opening size of the defect that would allow continuing erosion of the surrounding soil; and $D_{95}E$ = particle size adjacent to the open defect (i.e., envelope material) for which 95 percent by weight is finer after re-grading. This criterion assumes that the Foster and Fell (1999, 2001) continuing erosion criteria apply to erosion into an open joint, defect, or crack in conduits, walls, toe drains, or rock foundations, and that the crack width is equivalent to the filter opening size of the voids between the particles in a filter.

Again, it's important to remember that constrictions that are retaining soils and preventing erosion need to be continuous to some exit point. For example, bedrock joints/fractures need to be continuous to an open face and not covered by alluvium. In some rare cases where extensive void spaces may exist in coarse soils or bedrock, an open exit may not be needed, but sufficient "storage space" for eroded fines must be available. It is also important to consider the flow direction and likelihood of flow reversal.

D-6.9 Progression

Progression is the process of developing and enlarging an erosion pathway through the embankment core or foundation. The progression phase can be subdivided into three separate processes for concentrated leak erosion (scour) and backward erosion piping. These processes include: 1) formation of a continuous stable roof and/or sidewalls through the core; 2) the possibility that flows are limited by a constriction or an upstream zone or structure; and 3) the potential for an upstream zone to provide self-healing. These three considerations are commonly used, but other factors may also need to be considered for the progression phase in some cases. The progression phase includes all steps after continuation and prior to breach with the exception of intervention.

Enlargement of the erosion pathway may occur in either an upstream or downstream direction. For internal erosion mechanisms that do not necessarily require formation of a pipe that connects to the reservoir (i.e., stoping or internal migration), then the progression phase as defined here would likely be different. Currently there is no uniform practice for evaluation of progression for these other internal erosion processes, although they need to be included in specific events trees. For example, a standard progression event description could be modified to include "the probability that a large sinkhole forms in a critical area allowing progression to continue."

D-6.9.1 Progression – Continuous Stable Roof and/or Sidewalls

Formation of a continuous roof through the core or foundation is dependent on the soil conditions or presence of structures above the potentially erodible soils. Therefore, conduits, spillways, walls, and other concrete structures can form a roof along an identified potential internal erosion pathway. Interbeds of "hardpan," caliche, or other slightly cemented materials also constitute potential roofs for underlying soils that are not capable of supporting a roof by themselves. Absent these conditions, the capability of the soil to support a roof is dependent mainly on the properties of the soil above those being eroded.

Fell et al. (2008) summarized work by Foster (1999) and Foster and Fell (1999) that evaluated case histories and found that the two most important factors for roof formation are the fines content and whether or not the soil is saturated. Soils with fines contents greater than about 15 percent were found to be likely to hold a roof regardless of the plasticity (whether non-plastic or plastic). Other influential factors include the degree of compaction (loose soil less likely to support a roof) and reservoir operation (cyclic reservoir levels were more likely to cause collapse than constant levels). Research by Park (2003)[2] showed that sandy gravel with 5 to 15 percent

[2] Park's research was related to cracking in filters. Some of the test results were considered applicable to the potential for roof formation of soils.

non-plastic fines collapsed quickly when saturated. Park also found that sandy gravel with 5 percent cohesive fines collapsed after some time, but very slowly with 15 percent cohesive fines. Based on these studies, Table D-6-3 adapted from Fell et al. (2008), provides guidance on the likelihood a soil will be able to support a roof, absent overlying harder materials.

For concentrated leak erosion that occurs high in the embankment (e.g., cracks in the crest or a gap adjacent to a spillway wall), a roof is not necessarily a requirement for the process to progress. It is possible that the sidewalls could collapse and prevent further progression rather than collapse of a roof material. If the primary internal erosion mechanism is internal migration (stoping) without formation of a roof, then this event can be eliminated.

The presence of a structure or hard layer and soil properties are primary factors to consider in roof formation. Some other factors include soil variability along the seepage path, the length of the seepage path, and stress arching.

Table D-6-3 Probability of Holding a Roof (adapted from Fell et al. 2008)

USCS Soil Classification	Fines Content, FC (percent)	Plasticity of Fines	Moisture Condition	Probability of Holding a Roof (P_{PR})
Clays, sandy clays (CL, CH, CL-CH)	FC ≥ 50	Plastic	Moist or Saturated	0.9+
Silts (ML, MH)	FC ≥ 50	Plastic or Non-Plastic	Moist or Saturated	0.9+
Clayey sands, gravelly clays (SC, GC)	15 ≤ FC < 50	Plastic	Moist or Saturated	0.9+
Silty sands, silty gravels, silty sandy gravel (SM, GM)	15 ≤ FC < 50	Non-Plastic	Moist Saturated	0.7 to 0.9+ 0.5 to 0.9+
Granular soils with some cohesive fines (SP-SC, SW-SC, GP-GC, GW-GC)	5 ≤ FC < 15	Plastic	Moist Saturated	0.5 to 0.9+ 0.2 to 0.5
Granular soils with some non-plastic fines (SP-SM, SW-SM, GP-GM, GW-GM)	5 ≤ FC < 15	Non-Plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils (SP, SW, GP, GW)	FC < 5	Plastic	Moist or Saturated	0.001 to 0.01
		Non-Plastic	Moist or Saturated	0.0001

Notes:
(1) Lower range of probabilities is for poorly compacted materials (i.e., not rolled), and upper bound is for well compacted materials.
(2) Cemented materials give higher probabilities than indicated in the table. If the soils are cemented, use the category that best describes the particular situation.

The probabilities should not be used directly in a risk analysis, but rather used to help develop a list of more likely and less likely factors during an elicitation of probability estimates.

D-6.9.2 Progression – Constriction or Upstream Zone Fails to Limit Flows

There are some cases where internal erosion can progress to the point where the dam core or foundation is eroded through, but a flow constriction at some point along the path, an upstream zone, or facing element limits the flow from the reservoir to the point where erosion is arrested and a breach will not form. This is contingent upon the upstream zone being stable under the flows and having small enough openings to limit flows through the zone to levels that would prevent further erosion of the core. In essence, the flow is limited so that shear stresses are insufficient to detach soil particles.

Fell et al. (2008) suggest that the success of the upstream zone in limiting flows is highly dependent on whether the mechanism leading to a flaw in the core is also present in the upstream zone, with its ability to support a roof or crack of secondary importance. If the potential for the flaw to extend through the upstream zone is high and the potential for the upstream zone to support a roof or crack is high, then flow limitation is unlikely.

Examples of constrictions may include concrete or sheet pile walls within the embankment or that fully penetrate foundation soils greatly increase the likelihood of flows being limited. Modern concrete walls (crossing the internal erosion pathway, typically extending into rock) that are in good condition have the best chance for success. Steel sheet pile walls may be less effective under poor driving conditions or poor construction techniques. Concrete or steel membranes, soil-cement slope protection, geomembranes, or other linings on the upstream face of the dam can be effective in limiting flows, depending on their condition, but potential erosion of the underlying support for the facing may be an issue.

For failure modes that involve seepage paths through bedrock discontinuities, the flow could be limited by the aperture of those discontinuities. Similarly, failure modes in which the seepage flows into a crack or joint in concrete, such as an outlet works conduit, the flow may be limited. However, flow velocities could be quite high, which could lead to stoping (internal migration).

For potential failure modes through the foundation, upstream fine-grained blankets beneath and around the dam may not prevent initiation of erosion but may be effective in limiting progression. Flow limitation may occur due to an increase in head loss across the upstream blanket after uplift of the downstream blanket and initiation of erosion.

In unusual cases, progression could create a large enough void that results in failure of the structure or zone providing the constriction.

D-6.9.3 Progression – No Self-Healing Provided by Upstream Zone

Crack-filling action requires a granular zone upstream of the core with particles of a size, which can be transported by water flowing into the crack or pipe, and a downstream filter/transition zone or rockfill, which is sufficiently fine to act as a filter to these particles and the core.

Upstream granular zones have been observed to help supply crack-filling materials and contribute to self-healing. Typically, sinkholes appeared above the upstream filter/transition zone which is considered to be evidence of material being washed into the crack or pipe. Crack-filling action is only possible for central and sloping core earth and rockfill (or gravel shoulders) dams. The effectiveness of the crack-filling action depends on the compatibility of particle sizes of the granular material upstream of the core and in the downstream filter/transition zone, and then the compatibility of the downstream filter/transition material (with the washed-in particles) and the core. The internal erosion process may be arrested and not lead to breach if the crack or pipe progresses through the core, but there is an upstream zone which can collapse into it (i.e., the upstream zone is not capable of supporting a crack or a roof) and a downstream filter/transition zone which then acts as a filter. The washed-in materials aid in the filtering action against the downstream filter/transition zone, especially in cases of poor filter compatibility between the core and downstream filter/transition zone due to a lack of sand-sized particles in the core. In these cases, the probability of continuation may be high, but the washed-in particles may be capable of filtering against the downstream filter/transition zone reducing the potential for the pipe enlarging. There is less benefit when the washed-in particles are of similar sizes to the core material. There is limited benefit when there is no downstream filter/transition zone. The likelihood of success is difficult to estimate, but probably increases with thicker upstream zones, the presence of truly cohesionless materials, a variety of particle sizes, and the presence of a downstream shell or zone that will provide a filter for these materials that wash into and through the core. Finally, the size and nature of the defect in the core is a consideration (i.e., self-healing may occur early when the defect is a crack or later when the defect is a pipe).

Consideration should be given to whether the self-healing will occur early when the defect is small. In general, it is more likely to self-heal earlier in the process when sand size particles could be carried to downstream zone by relatively low flows. Gravel and larger sizes need high flows to be transported, so by the time flows are large enough to transport these sizes, significant enlargement of the erosion pathway may have already occurred. A well-documented example of this type of self-healing is in a case history for Matahina Dam in New Zealand (Gillon). Self-healing has also been observed at Suorva Dam in Sweden (Nillson 2005, 2007) and at Uljua Dam in Finland (Kuusiniemi 1991).

D-6.9.4 Unsuccessful Intervention

This event considers the likelihood that human efforts to detect and stop (or slow) the internal erosion process from breaching the embankment fail to work. This single event evaluates the potential that two components might occur: 1) detection (i.e., whether, or when, a developing failure mechanism would be observed and recognized as a problem); and 2) the ability to successfully intervene (i.e., can mitigating efforts be implemented in time to stop or slow the failure process to the point where dam breach does not occur?). The probability of unsuccessful intervention is typically captured in one event (but could be decomposed into two events of detection and unsuccessful intervention), just before breach, although it is recognized that intervention efforts could occur during all phases of an internal erosion process. When estimating the probability of unsuccessful intervention, it is acceptable to consider factors that would support earlier intervention, chronologically before the failure mode has completed the "progression" events.

Risk estimates must give due consideration for intervention actions. In order for intervention to be successful, the failure path must be detected, and repairs or lowering the water level must be performed prior to breach development. In some cases, it is useful to calculate the risk estimates for both with and without intervention to understand the potential for detection and the benefits of intervention, while at the same time not masking the seriousness of the issue by using intervention to reduce the estimated risk. This may be useful in making the case for additional monitoring or other actions to better understand or reduce the risks. Some levees are designed with planned intervention to achieve acceptable performance.

Experience in the dam safety community indicates that many internal erosion incidents progressed for decades (although they were not recognized as such early on), and that if detected, there is a high rate of successful intervention. For example, only about 1 percent of

Reclamation's incidents involving the initiation of internal erosion have led to dam breach (Teton Dam), and within USACE no incidents have resulted in dam breach. However, USACE has experienced several levee breaches. This success is due to a number of factors involving detection and successfully intervening at various points within the internal erosion process; however, two factors appear to particularly stand out in the tabulated cases.

- First, in most cases, signs of the potential initiation of internal erosion (e.g., sinkholes, sand boils, and excessive seepage) were observed and necessary remedial actions were quickly taken. Internal erosion incidents have typically been discovered by visual observation, sometimes by the public. For this reason, "eyes on the dam" is a key consideration. Is the dam or levee in a remote location? Are likely downstream exit paths observable (consider rockfill, tailwater, marsh areas, beneath blankets, etc.)? How often is the dam or levee visited/observed? How close does the public get? Are local officials (police, park rangers, and recreation staff) trained in dam and levee safety? Few cases have been detected by routine instrumentation monitoring, although it has happened. Over the long-term, piezometer and seepage measurement trends can be indicative of slowly developing internal erosion failure modes.

- Second, there are a number of instances where it appears that self-healing or collapse of developing internal erosion took place and either stopped the process or provided warning such that intervention could take place. This episodic nature of internal erosion incidents, which can lead to these failure mechanisms taking decades to progress (or initiate in some cases), has been demonstrated in all categories of internal erosion, particularly in those involving foundation materials, conduits, or drains. The episodic nature has the benefit of increasing the likelihood of observation, but it can be dangerous because a connection to the reservoir can be sudden, after progressing undetected for a long time.

Evaluating factors related to detection and physical intervention actions is very site-specific and requires judgment and subjective probability estimates (see Chapter A-6 Subjective Probability and Expert Elicitation). For example, if there is coarse rockfill on the downstream slope or ponding at the toe of the dam, it may be very difficult to detect new muddy seepage. If the reservoir is large and the release capacity is small, attempting to draw the pool down may be of little help. If equipment and materials are not readily available from nearby sources, there may be little that can be done in the way of emergency repairs. These are important items to consider when evaluating detection and intervention. If this event is estimated to have a high likelihood for success, it should be highlighted in the documentation, as this is critical information for the operations of the facility. Even if the estimated likelihood of success is low, it should still be pre-planned and attempted should it occur.

Fell et al. (2001, 2003) studied case histories of failures and accidents for internal erosion in the embankment, foundation, and embankment into the foundation. Based on the case histories and an understanding of the physical processes, they developed guidance on the time for progression beyond when a concentrated leak is first observed and development of a breach. Tables D-6-4 to D-6-6 are based on that study. In these tables, the qualitative terms for rates are defined in Table D-6-7. Table D-6-4 could be used to estimate the approximate time to dam failure after a concentrated leak is first observed. Most of the case studies were for breach by gross enlargement. Therefore, the method is only applicable to cases where the breach mechanism is gross enlargement. It is considered reasonable where the final breach is by slope instability, following development of a pipe. It will probably underestimate the time for breach by sloughing since breach by sloughing is a slowly developing mechanism which could take days to weeks to lead to breach. Breach by sinkhole development is potentially a rapid process in the final stages when the sinkhole emerges into the reservoir, but limited case history data exists.

Table D-6-4 Rate of Erosion of the Embankment Core or Foundation Soil (adapted from Fell et al. 2001, 2003)

Factors Influencing the Time for Progression and Breach				Approximate Likely Time (Qualitative)	Approximate Likely Time
Ability to Support a Roof	Rate of Erosion (Table D-6-5)	Upstream Flow Limiter	Breach Time (Table D-6-6)		
Yes	R or VR	No	VR or R–VR	Very Rapid	< 3 hours
Yes	R	No	R	Very Rapid to Rapid	3 to 12 hours
Yes	R–M	No	VR	Rapid	12 to 24 hours
Yes	R	No	R–M		
Yes	R	No	M or S	Rapid to Medium	1 to 2 days
Yes	R or R–M	No	M or M–S		
Yes	M or R–M	Yes	R or R–M		
Yes	M or R–M	No	S	Medium	2 to 7 days
Yes	R–M or M	Yes	S		
Yes	M	Yes or No	S	Slow	Weeks, even months to years

Table D-6-5 Rate of Erosion of the Embankment Core or Foundation Soil (used in Table D-6-4) (adapted from Fell et al. 2008)

Soil Classification	(I_{HET})	Time for erosion in the core of the embankment or in the foundation	
		0.2-gradient along pipe	0.5-gradient along pipe
SM with < 30% fines	< 2	Very Rapid	Very Rapid
SM with > 30% fines	2 to 3	Very Rapid	Very Rapid
SC with < 30% fines	2 to 3	Very Rapid	Very Rapid
SC with > 40% fines	3	Rapid	Very Rapid
ML	2 to 3	Very Rapid to Rapid	Very Rapid
CL-ML	3	Rapid	Very Rapid
CL	3 to 4	Rapid	Very Rapid to Rapid
CL-CH	4	Rapid	Rapid
MH	3 to 4	Rapid	Very Rapid to Rapid
CH with LL < 65	4	Rapid to Medium	Rapid
CH with LL > 65	5	Medium to Slow	Medium

Note: I_{HET} is the index value from the Hole Erosion test (HET)

Table D-6-6 Influence of the Material in the Downstream Zone of the Embankment on the Likely Time for Development of a Breach due to Gross enlargement of a Pipe (used in Table D-6-4) (adapted from Fell et al. 2003)

Material Description	Likely Breach Time
Coarse-grained rockfill	Slow – Medium
Soil of high plasticity (PI > 50) and high clay-size content including clayey gravels	Medium – Rapid
Soil of low plasticity (PI < 35) and low clay-size content, all poorly compacted soils, silty sandy gravels	Rapid – Very Rapid
Sand, silty sand, silt	Very Rapid

Table D-6-7 Qualitative Terms for Times of Development of Internal Erosion and Breach (adapted from Fell et al. 2003)

Qualitative Term	Equivalent Time
Slow (S)	Weeks or months, even years
Medium (M)	Days or weeks
Rapid (R)	Hours (> 12 hours) or days
Very Rapid (VR)	< 3 hours

D-6.9.5 Assessing the Rate of Enlargement of a Pipe

The time for erosion to progress is an important factor for assessing the likelihood of successful intervention and is dependent on the soil erosion properties. In addition, breach mechanisms vary in their time to fully develop and catastrophically release the impounded water. Therefore, the likelihood of successful intervention should also consider the potential time available based on the breach mechanism being considered.

However, the duration of the critical loading is also an important condition, and episodic cycling of the reservoir may result in the progression of erosion occurring only sporadically (at high pool) over the course of weeks, months, years or even decades. This can complicate the assessment of the rate of enlargement significantly.

Procedures detailed in Appendix D-6-I may provide useful insights into the development time for erosion progression. However, they not include the effects of pool duration and reservoir cycling.

D-6.9.6 Breach

Breach is the fourth and final phase of internal erosion in which materials in the embankment are eroded, and the opening widens and deepens until an uncontrolled release of impounded water occurs. Additional downcutting or deepening may continue, and the breach enlarges or widens by side erosion and mass wasting of material from the banks of the developing breach. Breach occurs when either the failure mode is not detected or intervention is not attempted or is unsuccessful. If breach initiates due to internal erosion, it usually leads to complete failure. The full contents of a reservoir may not be lost depending upon many factors (e.g., breach located on an abutment shelf formed by non-erodible rock). Factors that reduce the likelihood of breach include large freeboard, large downstream rockfill zone, presence of a core wall (or similar feature) that remains in place, and a water level that drops below the inlet of a developing pipe before a breach mechanism has time to develop (e.g., reservoir has small storage capacity). The type of breach depends on the internal erosion mechanism being considered, embankment type, and the specific failure mode being considered. According to Fell et al. (2008), there are four breach mechanisms typically considered:

- **Gross enlargement of a pipe or concentrated leak:** If the erosion pathway or "pipe" connects to the impounded water, rapid erosion and enlargement of the pipe could develop until the crest collapses into the pipe. If the amount of crest drop is greater than the available freeboard, overtopping of the embankment could quickly lead to a breach. If overtopping does not occur, the embankment could be severely damaged, and breach could still occur by concentrated flow through cracks. If the likely breach mechanism for a potential failure mode is breach by gross enlargement, as opposed to sinkhole development or sloughing, a breach is generally more likely to occur. If the downstream or landside shell is unable to support a roof, sloughing or unraveling would be the more likely breach mechanism.

- **Sloughing or unraveling of the downstream face**: In situations where the downstream or landside zone is not capable of sustaining a roof, over-steepening of the downstream or landside slope due to progressive slumping can eventually lead to complete loss of freeboard. Soil particles are eroded, and a temporary void grows near the downstream or landside face until a roof can no longer be supported, at which time the void collapses. This mechanism is repeated progressively until the core is breached or the slope is over-steepened to the point of instability. Unraveling refers to progressive removal of individual rocks by large seepage flows through a downstream rockfill zone. According to Leps (1973), the stability of rockfill against through-seepage depends on the following characteristics and conditions (listed in increasing importance): specific gravity of the rock particles, dominant particle size of the rock fill, gradation and shape of the rockfill particles, relative density of the rockfill, rate of discharge, maximum gradient, and inclination of the downstream slope of the rock fill. Methods to evaluate the stable rock size as a function of unit discharge and downstream slope include Olivier (1967), Solvik (1991), and EBL (2005).

Reclamation's Fontenelle Dam in Wyoming nearly breached in 1965 by sloughing, but the breach process occurred slowly enough so that the reservoir water surface was able to be lowered over the span of several days and arrest the breach. In contrast, Hell Hole Dam, a rockfill structure in California, failed from overtopping during construction in 1964, but it handled a leakage of about 13 cfs/ft before small slides and erosion began to progressively occur at the toe. Once this began, failure occurred within about 3.5 hours (Leps 1973). The core of Reclamation's Minidoka Dam overtopped during construction (1904 to 1906), and the downstream rockfill zone withstood flows estimated up to 1,000 cfs. The water surface elevation was 8 feet below the normal water surface when the core overtopped.

- **Sinkhole development**: This mechanism refers to stoping of material upward, creating a sinkhole or depression in the embankment that compromises the embankment or lowers the crest below the impounded water level. For breach to occur, the sinkhole would need to be large enough to lead to overtopping. USACE's Wolf Creek Dam was constructed over karst features and has experienced numerous sinkholes. Due in part to concern that sinkholes may lead to potential breach, major mitigation measures have been completed.

- **Slope instability**: Internal erosion could cause high pore pressures in the foundation or embankment, resulting in reduced shear strength and slope failure. Breach could occur if the failure surface either intersects the impounded water level, or the slope deformations are significant enough that the remnant can't resist the water load. Although it is possible, this is generally not considered to be a very likely breach mechanism for most dams. No historical failures from slope instability due to increased pore pressures in the downstream slope are known to exist, and a unique set of circumstances would need to exist for it to be a major concern.

All four mechanisms lead to crest settlement and overtopping erosion. One or more of the mechanisms may occur during the breach process, and it is generally not necessary to know precisely which mechanism(s) would occur. However, risk estimates should typically be developed considering the most likely breach mechanism(s).

There are a few cases where once failure has initiated and progressed, and intervention has been unsuccessful, complete breach of the dam did not necessarily follow. Many Reclamation and USACE dams have large flood storage resulting in large normal freeboard. If the operative breach mechanism was stoping (forming a sinkhole near the crest) or progressive slumping and erosion at the toe of the dam during periods when the reservoir is low, the large freeboard may prevent failure by keeping the sinkhole above the reservoir surface, or by formation of a "berm" at the downstream slope from the slumped material that ultimately arrests breach development. In addition to large freeboard, other factors that have led to a reduced probability of complete breach include a concrete corewall to nearly full dam height (which is capable of retaining the reservoir even if a "pipe" or sinkhole develops). In the case of internal instability of core material, not only must the finer particles be washed through the coarser materials, but the remaining fraction must sustain enough flow such that it is also completely eroded. It is also possible that a small reservoir volume may empty through an opened seepage path before complete dam breach can occur. Breach mechanisms vary in their time to fully develop and catastrophically release the reservoir, and the intervention node should consider the potential time available based on the breach mechanism being considered.

D-6.9.7 Accounting for Uncertainty

Given the difficulties in quantifying seepage and internal erosion behavior, there is a high degree of uncertainty in the estimates. Sensitivity analysis or other appropriate uncertainty analysis methods can be used to explicitly show how uncertainty influences the risk estimate.

D-6.9.8 Relevant Case Histories

A summary overview of several key incidents is provided below, starting from early history to the present, illustrating that internal erosion can occur at virtually any time during the operational life of an embankment dam (Engemoen and Redlinger 2009).

D-6.9.8.1 Avalon Dam

Avalon Dam in New Mexico failed twice; once in 1893 from flood overtopping and later in 1904 from internal erosion. After the second failure, Avalon was taken over by Reclamation and reconstructed in 1907. Although this dam was not part of Reclamation's inventory when it failed, it was one of Reclamation's earliest dealings with an internal erosion incident. Avalon Dam was one of several dams built in the late 1800's or early 1900's that featured a rockfill downstream section which buttressed an upstream earthfill zone. It is notable that a number of failures or serious incidents occurred at other non-Reclamation dams having this similar configuration, including McMillan Dam, Black Rock Dam, and Fish Lake Dam. In all these cases, a seepage path existed through their earthfill zone that flowed down into underlying rockfill. The exact cause of failure of Avalon Dam is unclear, but explanations included piping of the embankment due to the severe incompatibility of the earthfill and rockfill from a filtering/retention perspective, or erosion at the base of the earthfill due to flows in the upper portion of the limestone foundation.

D-6.9.8.2 Fontenelle Dam

A very serious internal erosion incident occurred in 1965, when Fontenelle Dam nearly failed during first filling. Significant seepage traveled through the open jointed sandstone foundation rock, emanating 2,000 feet downstream in a low area as well as in the right abutment near the spillway. Seepage led to the erosion of more than 10,000 cubic yards of embankment materials before the intervention efforts of large outlet works releases and dumping of rockfill into the embankment erosion area eventually lessened the flows and the erosion. Fortunately, the large capacity outlet works was able to lower the reservoir by approximately 4 feet per day, quickly reducing the head at the abutment area where internal erosion was occurring. In less than 2 days of drawdown, the reservoir was lowered off of the spillway approach channel which undoubtedly was feeding seepage into the problem area. The primary cause of the near failure was thought to be inadequate grouting of the jointed sandstone and the lack of foundation treatment measures such as slush grouting and dental concrete, which led to seepage near the base of the dam that removed embankment material and led to the growth of voids and stoping. Contributing factors included the presence of infilling or soluble material in the jointed rock that may have inhibited grout travel; residual or redeposited soluble salts in the rock that may have reacted with the grout causing premature set or ultimate softening; the erodible nature of the embankment core material; and a steep right abutment that created difficulties in achieving good bond or contact between the embankment and abutment, encouraged differential settlement and cracking of the embankment, and made shallow grouting difficult because low pressures were required to prevent movement of the rock.

Another factor not mentioned in early reports was the unfavorable orientation of the abutment with respect to the potential for hydraulic fracturing. In hindsight, an obvious key factor in the near failure, in addition to the lack of sufficient foundation treatment, was the lack of an internal filter and drainage zone that would render seepage through both the foundation and embankment harmless with respect to the removal of soil particles and the buildup of pore pressures. A couple of key details are that the average zone 1 core material in the dam is reported as being a SC and CL with 13 percent plus No. 4 material and having a LL of 31 and a PI of 13. However, the core material remaining after the near breach in the abutment area was generally described as a well graded mixture of sandy gravel and silt. No crack in the core was noticed during close inspection of the piping channel through the zone 1. Zone 2 materials described as select sand, gravel and cobbles as well as the materials in the miscellaneous zone sloughed during this incident and an incident that occurred four months prior and were easily removed by the concentrated seepage.

D-6.9.8.3 Teton Dam

Teton Dam failed from internal erosion during first filling in 1976, marking the first and only failure of a Reclamation embankment dam. The failure was similar to the incident at Fontenelle Dam 11 years earlier, with excessive seepage through a highly jointed foundation rock leading to erosion of a highly erodible core material during initial reservoir filling. Contributing factors included a low permeability transition zone that contained too many fines to act as a drain for the core or serve as a filter, the lack of foundation filters on the downstream face of the cutoff/key trenches, insufficient treatment of the open joints in the rock foundation, the presence of a highly erodible core material, the rapid rate of initial reservoir filling, and an inoperable outlet works. Reports were prepared by an independent panel and a Government panel assembled to review the cause of failure.

There have been a number of reasons given as to how a defect in the core materials deep within the right abutment cutoff trench came about; some are as simple as resulting from the fill becoming frozen during winter shut down to more complex theories related to low stress zones and hydraulic fracturing caused by arching of the dam over the steep narrow cutoff trench. However, it is critical to recognize that the joints, fractures and openings in the downstream wall of the cutoff trench and the remaining foundation downstream of the trench were severely incompatible with respect to filtering and retention of the very fine grained, erodible core materials, as well as the silt infillings in some of the joints themselves. It would have been virtually impossible to construct a perfect core without defects to overcome these conditions, and the focus should have been on proper foundation treatment and filter protection for the core and the silt infillings.

D-6.9.8.4 Caldwell Outlet Works at Deer Flat Dams

The Caldwell Canal outlet works, with a capacity of 70 cfs, is a cut-and-cover conduit located through the left abutment section of the Upper Embankment at Deer Flat Dams in Idaho, and was completed in 1908. The foundation materials in the vicinity of the Caldwell conduit consist of mostly poorly graded sand and silty sand with some gravel. Caliche layers exist in some areas of the dam's foundation as well. A dam safety inspection in 2001 (93 years after construction) noted some sediments in the seepage from a crack in the conduit located 65 feet upstream of the outlet portal. A large sand deposit approximately 6 feet wide by 15 feet long and 10 to 12 inches deep was observed downstream of the outlet structure. Although speculated to be windblown materials, it was also judged possible that the observed sediments could have been materials transported into the conduit by seepage flows. Then, in 2004 sediment was observed at the base of a crack in the conduit approximately 125 feet downstream of the regulating gate. Subsequently, ground penetrating radar was utilized in the conduit, and potential anomalies were detected between 100 and 150 feet downstream of the gate. Follow-up drilling through the conduit revealed voids beneath the conduit varying from ½-inch to 5 inches in depth, presumably caused by internal erosion of foundation soils into or along the conduit. Piezometers installed below the conduit revealed consistently low pressures similar to tailwater levels beneath the conduit from the downstream portal upstream to within 20 feet of the intake. It was concluded that backwards erosion piping had occurred along most of the conduit, with potentially high gradients existing at the upper end of the conduit. A large upstream berm was constructed to minimize the potential for upstream breakout of the piping pathway to the reservoir, until permanent corrective actions could be taken.

In the case of Upper Deer Flat Dam, even though in general there are gravels present in the embankment fill as well as the foundation, gravel sized particles were found to be absent over a large extent of the conduit foundation during the re-construction. Even if coarser particles were present in the soil mass, the mechanism of a soil filtering against a crack in the bottom of a conduit can be complicated by the fact that a flow path beneath the structure will not necessarily transport coarser particles up into or against the crack in the bottom of the conduit. Any particles transported to such a crack may drop away during times of lower gradients such as under lower reservoir operating conditions. Therefore, caution against the use of liberal filter/retention criteria in such a case is advised.

D-6.9.8.5 A.V. Watkins Dam

A.V. Watkins Dam (formerly known as Willard Dam) is a U-shaped (in plan view) zoned earthfill structure constructed within Willard Bay of the Great Salt Lake. Constructed from 1957 to 1964, the dam is 36 feet high at its maximum section and slightly more than 14.5 miles long. Upon first filling of the reservoir in 1965, as the reservoir reached within approximately 2 feet from full, numerous wet areas (with some areas displaying quick conditions) appeared at the downstream toe of the dam. After this discovery, filling of the reservoir was halted, the reservoir was lowered and a toe drain was constructed approximately 15 feet from the downstream toe from 4 to 5 feet deep in the foundation, consisting of 8-inch diameter bell and spigot concrete pipe with open joints and surrounded by gravel. Toe drain outfalls were constructed at approximate 1,000-foot intervals to discharge into the South Drain; a long open ditch excavated about 130 feet downstream of the dam toe to help drain farm land as well as seepage. The toe drain was apparently successful in drying up the downstream toe area and the reservoir was eventually filled to the top of active conservation water surface.

In November of 2006, A.V. Watkins Dam nearly failed at a location in the same general area that created problems during initial filling, as the result of piping and internal erosion of the foundation soils. Two days previously, a local cattle rancher working just downstream of the incident area noticed seepage and some silty material exiting from the cut slope of the South Drain. The rancher continued to observe the seepage and erosion into the South Drain until Monday, November 13, when he became concerned over the increase in seepage and the appearance of what he described as “dark clay” exiting into the South Drain. He called authorities and Reclamation began 24-hour monitoring and initiation of an emergency drawdown of the reservoir. Piping of the foundation soils was occurring from beneath the dam below a somewhat continuous downstream, but absent upstream, series of thin hardpan layers, and the fine-grained, silty sand soils were exiting from the dam's downstream toe and from the base of the north slope of the South Drain. Approximately 140to190 gallons per minute of seepage was exiting from sand boils at the downstream toe of the embankment (but upstream of the toe drain), flowing across the ground surface and into sinkholes between the toe of the embankment and the South Drain. The seepage appeared to be re-emerging at the base of the bank of the South Drain and was depositing large amounts of sand into the South Drain. Figure D-6-22 depicts the conditions.

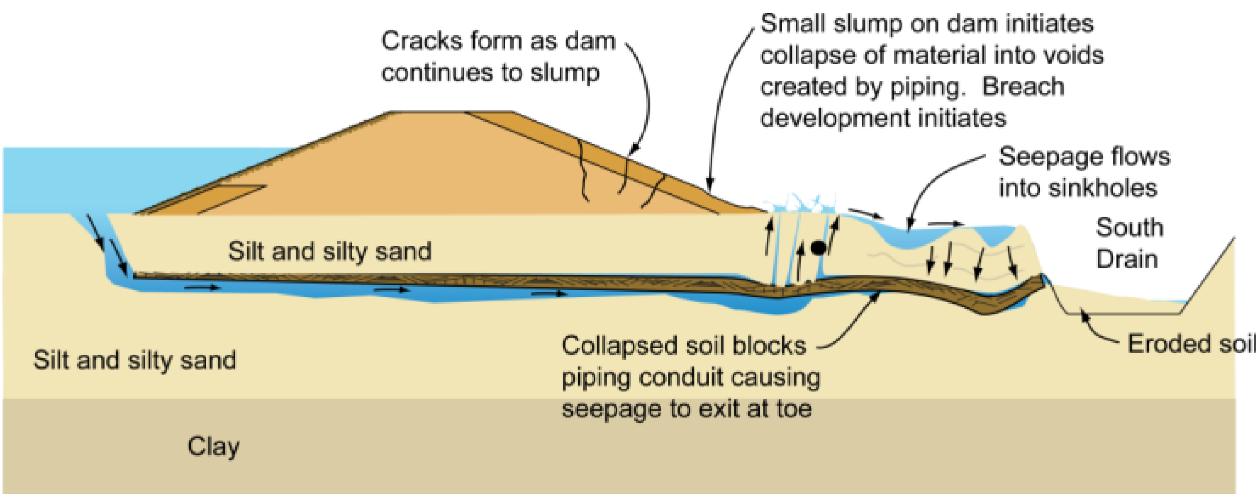


Figure D-6-22 Failure Mode In-Progress (not to scale)

Efforts to save the dam focused immediately on transportation of filter sand and gravel materials to the site to begin placement of these soils directly over the sand boils in an attempt to stop the erosion of foundation soils. Initially filter sand was placed over the sand boils but that was quickly washed away due to the high exit velocities. Gravels were then placed over the sand boils until the exit velocities were reduced enough to allow placement of the filter sand. This reduced the flow and erosion of soil enough to allow the placement of a sufficiently large berm consisting of additional filter material, drainage material, and minus 5-inch pit-run material, to counter the uplift pressures in the emerging seepage at the toe.

On November 16, Reclamation technical staff determined the failure mode was still in progress and additional remedial action was required. It was noticed that seepage and erosion was still occurring into the South Drain. On November 17 and 18, a berm was added to the upstream slope of the embankment extending into the reservoir to stem the flow of the water into the foundation and any inlets to potential piping channels (located just beyond the upstream dam toe and within the riprap) that were postulated to be the sources for the concentrated seepage entering the foundation. These efforts were successful in stopping the foundation erosion and immediately reducing the overall seepage flows. Some key lessons learned at this dam to consider in future dam designs and risk analysis are:

- Internal erosion can initiate, progress and nearly fail a dam with an erodible foundation at very low head to seepage length ratios, in this case generally about 0.09 (locally may have been about 0.06 due to rodent holes), if the exit point for the unfiltered seepage is nearly horizontal, the soil is highly erodible, and a roof is present.

- Rodent activity can suddenly aggravate a meta-stable seepage situation, as rodents can fairly quickly excavate a burrow and shorten a potential seepage path, compared to the more gradual particle transport caused by seepage at these low gradients.

- Construction of open trenches downstream of the toe of a dam provides a location into which materials can be eroded.

- Toe drains installed as the primary defense against foundation internal erosion, especially when the drain is installed after an occurrence of piping was observed, can be critical to the performance of the structure. Plugging of the toe drains appeared to have been occurring at this site. It is not clear that the toe drain plugging was a significant contributor to the occurrence of the incident.

- Changes to seepage conditions that occur over a long period of time can be difficult to recognize and the knowledge about the presence of buried drains can be lost. Consideration should be given to estimating risks for (or at least considering as a potential failure mode) every location that seepage or wet spots are known to exist at a dam, as well as those areas typically analyzed (see also Bliss and Dinneen 2007).

D-6.9.8.6 Stilling Basin at Davis Creek Dam

Davis Creek Dam is a modern embankment dam in Nebraska, completed by Reclamation in 1990. A sinkhole was reported adjacent to the outlet works on May 11, 2007. The sinkhole was located against the left side of the outlet works control house immediately upstream of the stilling basin, and measured approximately 5 feet along the wall, 2 feet wide away from the wall and about 6 feet deep. The sinkhole was located in the structural backfill composed of fine to medium sands. The perimeter of the sinkhole was probed with a steel rod, which could be inserted with ease vertically along the wall in the sand to a depth of about 10 feet below the bottom of the sinkhole or about 16 feet below the original ground surface. Subsequent video inspections of the spillway underdrain system found sand in the drain pipes. Due to a defect in the underdrain system, whether from broken pipe or inadequately constructed filters, structural backfill, filter sand, and possibly foundation sands were being internally eroded into the underdrain system and then removed downstream by the action of the drains during outlet works operation. A grouting operation was subsequently undertaken, and it took more than 20 cubic yards of grout to fill voids beneath the stilling basin and surrounding areas. The precise location and lateral extent of the void system could not be defined, and it is uncertain if the erosion had progressed upstream along the outlet works beyond the limit of the upstream edge of the sinkhole. A filtered drainage system was also constructed around the sides of the basin to encourage drainage and thus reduce uplift pressures.

The underdrains were installed to assist in preventing floatation of the stilling basin structure both during dewatering of the stilling basin and during operations should the hydraulic jump move downstream. They were constructed such that during certain operating conditions outlet works discharges running by the drain outlets created low pressures thus resulting in drain flow and lowered uplift pressures. Vents were installed to ensure negative pressures did not develop. This fairly sophisticated drain system, if damaged, can apparently be very efficient in causing particle transport from the foundation. Since the operations of the outlet are intermittent, removal of soil would be intermittent and could occur over a long period of time. The typical winter seepage regime could have primed the system with water and soil particles and then the underdrains could have nearly instantaneously removed the water and some soil from beneath the structure each year under certain operating conditions. Hydraulic connection of the stilling basin to the groundwater was potentially causing very severe transient seepage conditions and particle transport.

D-6.9.8.7 Wister Dam

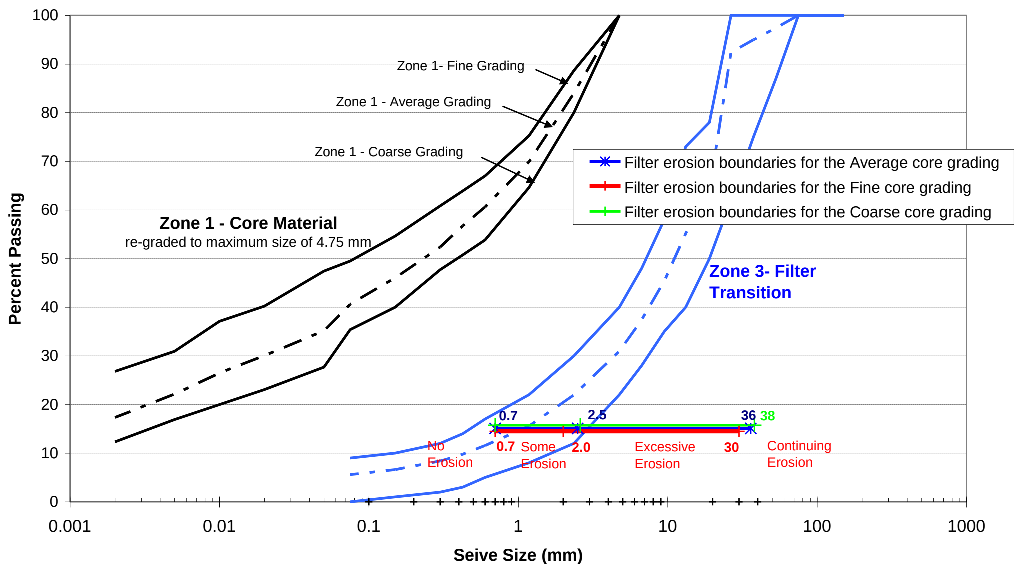
Wister Dam, Oklahoma was a severe case of differential foundation settlement likely causing cracks in the embankment (flaws) that skewed across the dam aligned with soils left in the foundation resulting in concentrated leaks and scour. Concentrated leakage was not observed until the reservoir rose above the entrance to the cracks which supports the need to consider whether or not flaws exist above historic maximum pools. The amount of leakage was increasing while the reservoir was being lowered indicating that erosion was continuing and progressing. The design did not provide a defense for either of these phases at the location the cracks existed as the scoured material was transported completely through the embankment above the ineffective drainage blanket. Intervention included lowering the reservoir below the entrance to the cracks which caused leakage to cease thus preventing further damage and possible failure of the dam. Embankment soils were later found to be dispersive which indicates they were likely very erodible. The differential head to estimated seepage path length along the skew was estimated to be about 1/50 or 0.02. Sherard used the term "concentrated leakage" to describe the flow out the downstream face of the embankment.

D-6.9.8.8 Pishkun Dikes

Pishkin Dikes is a case of internal migration. In June 1997 the Greenfields Irrigation District staff discovered a sinkhole adjacent to the concrete air shaft associated with the outlet works at Dike 4. Subsequent investigations revealed that embankment materials were moving into the outlet works conduit and the sinkhole had grown to about 12 feet in depth and extended beneath the outlet works gate house structure. The problem was traced to structural failure and collapse caused by deterioration of the 6-inch-thick concrete walls at the bottom of the air shaft. Freeze-thaw and a possible poor first lift of concrete were conjectured as the reason the concrete failed. The opening in the concrete allowed embankment material to migrate into the shaft which connects into the outlet works conduit area located immediately downstream of the slide gates. The sinkhole was excavated, the air shaft was filled with lean concrete and a new pipe was installed to supply air to the downstream side of the gates. The embankment was repaired witha processed sand and gravel filter material.

D-6.9.9 Example

Given the gradation curves shown in the Figure D-6-23, estimate the probability of no erosion, some erosion, excessive erosion, and continuing erosion for the fine, average, and coarse Zone 1 base soil gradations. Assume the representative gradations of the re-graded base soil corresponds to 90 percent all gradation tests.



Assessment of Zone 1 core against no erosion, excessive erosion and continuing erosion criteria

Core Gradation	Base soil sizes (mm)				No Erosion	Excessive Erosion	Continuing Erosion
	DB85 (mm)	DB95 (mm)	% passing 0.075mm	% fine-medium sand (0.075 - 1.18mm)	DF15 (mm)	DF15 (mm)	DF15 (mm)
Fine Grading	1.9	3.3	50	25	0.7	2	30
Average	2.4	4	41	29	0.7	2.5	36
Coarse Grading	2.5	4.2	35	30	0.7	2.6	38

Figure D-6-23 Example

D-6.9.10 References

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Appendix D-6-A: Large Dam Failure Statistics

Table D-6-A-1 Overall Statistics of Embankment Dam Failures (adapted from Fell et al. 1998, 2000)

No. of Cases		% of Failures (if Known)		Average Probability of Failure	
All Failures	Failures in Operation	All Failures	Failures in Operation	All Failures	Failures in Operation
Internal Erosion through the Embankment					
39	38	30	33	3.5E-03	3.5E-03
Internal Erosion through the Foundation					
19	18	15	15	1.5E-03	1.5E-03
Internal Erosion from the Embankment into the Foundation					
2	2	1.5	1.5	2.0E-04	2.0E-04

Table D-6-A-2 Historical Frequencies of Failures and Accidents (adapted from Fell et al. 1998, 2000)

Case	Total	In Embankment	Around Conduits and adjacent to Walls
Internal erosion failures	36	19	17
Internal erosion accidents	75	52	23
Seepage accidents with no detected erosion	36	30	6
Total number of failures and accidents	146	101	46
Population of dams	11,192	11,192	5,596
Historical frequency for failures and accidents	0.013	0.009	0.0082
Proportion of failures and accidents on first-filling	36%		
Proportion of failures and accidents after first-filling	64%		
Historical frequency for first-filling		0.0032	0.0030
Historical frequency after first-filling		0.0058	0.0052
Historical annual frequency after first-filling		2.2E-04	2.0E-04

Table D-6-A-3 Time of Incident for Internal Erosion through the Embankment (adapted from Fell et al. 1998, 2000)

Time of Incident	No. of Cases		% of Cases (if Known)	
	Failures	Accidents	Failures	Accidents
During construction	1	0	2	0
During first-filling	24	26	48	26
After first-filling and during first 5 years	7	13	14	13
After first 5 years	18	60	36	61
Unknown	1	3	–	–
Total	51	102	100	100

Table D-6-A-4 Time of Incident for Internal Erosion through the Foundation(adapted from Fell et al. 1998, 2000)

Time of Incident	No. of Cases		% of Cases (if Known)	
	Failures	Accidents	Failures	Accidents
During construction	1	0	5	0
During first-filling	4	23	20	30
After first-filling and during first 5 years	10	19	50	24
After first 5 years	5	36	25	46
Unknown	1	7	–	–
Total	21	85	100	100

Table D-6-A-5 Incidents of Cracking and Hydraulic Fracturing(adapted from Fell et al. 2008)

Cracking Mechanism	No. of Cases	Percentage
Differential settlement, cross-valley	20	35
Differential settlement, cross-section	6	11
Differential settlement, foundation	4	7
Differential settlement, embankment staging	0	0
Desiccation cracking	3	5
Closure section	3	5
Total	36	63

Table D-6-A-6 Incidents of Poorly Compacted and High Permeability Zones (adapted from Fell et al. 2008)

Location	No. of Cases	Percentage
At the foundation-embankment interface	5	9
In the embankment	16	28
Total	21	37

Appendix D-6-B: Historical Frequencies

Reclamation researched the frequency of internal erosion incidents for their portfolio of over 230 high and significant hazard embankment dams. A summary of Reclamation's research and experience is discussed in the following section on initiation.

Use of Historical Frequencies

Estimating the probability of an internal erosion failure is very difficult and lacks deterministic approaches. Thus, thus the use of similar case histories provides some degree of "ground truth" or empiricism/precedence to the evaluation. Laboratory testing of small specimens to develop erosion properties and similar data may not be representative of the weak link or true condition in the spatially large embankment-foundation system. Similarly, seepage models may not be representative of the key hydraulic conditions that would drive the development of an internal erosion PFM along a long seepage path through variable materials. It may not be a wise use of limited funds to spend significant monies in an effort to estimate a probability that is arguably no more than an index value.

Estimated historic rates of internal erosion initiation can provide risk teams with a relative range or average value for various types of internal erosion (i.e., a "starting" or "anchoring" point). Reclamation in the past has typically based the likelihood of this event on the documented historical rate of internal erosion failures and incidents (specifically work by the UNSW), and adjusted upward or downward based on site specific factors. Most recently, reviews of Reclamation internal erosion incidents have been made (Engemoen and Redlinger 2009; Engemoen 2011, 2017). The following is a discussion from those studies:

- Reviews of Reclamation internal erosion incidents indicate there have been a total of 97 known incidents and one failure. Internal erosion incidents have occurred throughout the history of Reclamation embankments, and sometimes multiple instances at the same dam. The total number of dams that have experienced incidents is 60, or about 1 in every 4 Reclamation embankments.

- ***These incidents are not limited to first filling but can occur at any time in a dam's life.*** About 30 percent of Reclamation incidents have occurred during the first five years of reservoir operation, and 70 percent of all incidents have occurred after more than five years of successful operation.

- The incidents have also not been limited to older or deteriorated dams; newer dams have also had incidents. Approximately half of the incidents occurred in dams that were more than 25 years old, and the other half in dams that were less than 25 years old.

- Each incident was classified into one of five categories: 1) internal erosion through the embankment; 2) internal erosion through the foundation; 3) internal erosion of embankment into foundation; 4) internal erosion into or along a conduit; and 5) internal erosion into a drain.

- In addition, each incident was also classified into one of four internal erosion mechanisms: 1) backward erosion piping; 2) internal migration (formerly called progressive erosion); 3) scour; and 4) suffusion/suffosion (related to internal instability). Admittedly, the assignment of an internal erosion mechanism to a past incident requires a lot of judgment – in many cases a definitive understanding of just what type of process or mechanism is not clear. Furthermore, some incidents may well involve a combination of mechanisms.

- For both classification exercises, the evidence for developing internal erosion is also shown, as either “excessive seepage” or “particle transport.” The use of particle transport was limited to those cases where clear evidence of internal erosion was noted, such as the presence of sinkholes, voids, sand boils that were moving soils, or turbid seepage water. Of the total 97 incidents/failures at Reclamation embankments, there have been a total of 66 cases where particle transport was observed.

- The following two tables portray the incidents in two different ways; first by category (location), and secondly by type of mechanism.

Table D-6-B-1 Category of Internal Erosion Incidents at Reclamation Embankments

Category of Internal Erosion	Incidents/Failures with Definitive Particle Transport	Incidents with Excessive Seepage and Perhaps Sand Boils	All Incidents and Failures
Embankment only	5	4	9
Foundation only	28	16	44
Embankment into foundation	7	11	18
Into/along conduit	8	0	8
Into drain	18	0	18
Total	66	31	97

The following observations from Table D-6-B-1 are of note.

- Approximately 45 percent of the tabulated internal erosion incidents have involved internal erosion through the foundation, likely due to the significant number of Reclamation dams without a fully penetrating cutoff over their entire length, the pervious nature of the foundation materials leading to significant seepage, and the presence of erodible soils in the foundation.

- Of the 44 foundation incidents, 10 involved glacial soils, and 5 were attributed to bedrock seepage. The remainder, or majority, of the incidents were in soil foundations comprised of alluvium, colluvium, eolian, or landslide deposits.

- When considering both internal erosion through the foundation and internal erosion from embankment into foundation, the foundation has played a role in at least two-thirds of all Reclamation incidents.

- The relatively low frequency of internal erosion through the embankment incidents might be explained by Reclamation's use of wide cores (long seepage path) often flanked by shells of sands/gravels/cobbles (providing some filtering capability). In addition, nearly 60 percent of Reclamation embankment cores are comprised of plastic soils (PI$\geq$7) and another 7 percent feature an impermeable reservoir liner or cutoff wall.

- The relatively high number of internal erosion into drain incidents may be due to decades of relatively poor design details for drains (open jointed pipe, brittle pipe materials, coarse gravel envelopes, and thin filters).

Table D-6-B-2 Postulated Internal Erosion Failure Mechanisms Involved in Incident

Category of Internal Erosion	Incidents/Failures with Definitive Particle Transport	Incidents with Excessive Seepage and Perhaps Sand Boils	All Incidents and Failures
Backward Erosion Piping	16	6	22
Internal Migration	24	0	24
Scour	17	20	37
Suffusion/Suffosion	9	5	14
Total	66	31	97

- The following observations from Table D-6-B-2 are of note.

    - One-half of all incidents are suspected to have involved internal migration or backward erosion piping, with each mechanism accounting for about a quarter of the total.

    - Scour is believed to account for approximately 40 percent of the total incidents.

    - Suffusion/suffosion is believed to account for about 1/7 of the total incidents.

  - Of these, half involved glacial soils.

    - ***The vast majority (~80%) of incidents involved cohesionless or low plasticity soils (PI < 7).***

The following table portrays the age of the dam (or modifications to a dam) at the time of each incidents.

Table D-6-B-3 Age of Dam at Incident and Mechanism Type

Dam Age at Incident	No. of Piping Incidents	No. of Internal Migration Incidents	No. of Scour Incidents	No. of Suffusion-Suffosion Incidents	Total No. of Incidents
≤ 5 years	6	5	14	4	29
6-15 years	1	2	5	0	8
16–25 years	5	5	2	0	12
26-35 years	2	1	3	1	7
36-45 years	3	2	3	1	9
46-55 years	0	3	2	3	8
56-65 years	2	1	3	0	6
66-75 years	0	3	4	2	9
76-85 years	2	2	0	2	6
> 85 years	1	0	1	1	3
Totals	22	24	37	14	97

- The following observations from Table D-6-B-3 are of note.
    - Incidents are much more likely to occur in the first five years of reservoir operation (which includes first filling of the reservoir).
    - However, incidents continue to occur beyond 5 years, with little significant decline in rate of incidents after 5 years.
    - Most (60 to 75%) of the incidents involving internal migration, scour, and suffusion/suffosion occur in the first 25 years of operational history.

- However, piping incidents tend to occur throughout the operational history; i.e., they are as likely to occur late as early.

Influence of Reservoir Level at Time of Incident

There is a widely held belief that most internal erosion incidents in embankments or foundations initiate at threshold reservoir levels that create sufficient hydraulic gradients or velocities to begin eroding susceptible soil particles. Thus, practitioners are likely to be more concerned about internal erosion at high reservoir levels. In fact, based on Reclamation incidents, internal erosion can manifest at varying reservoir levels.

A special condition involving reservoir levels involves the initial filling of the reservoir. Initial filling refers to the first time, shortly after completion of construction, that the reservoir is filled to its normal operational level (frequently the top of active conservation capacity at Reclamation reservoirs). Other studies have pointed out the frequency at which embankments experience internal erosion incidents during first filling. At Reclamation, 23 of the 97 incidents, or about 24 percent, occurred during initial filling of the pool. This points out the need for careful surveillance during initial reservoir filling (or re-filling after a modification).

However, at Reclamation, 74 incidents occurred during operations after first filling. The following table portrays the level of the reservoir at the time of these 74 observed incidents at Reclamation embankments.

Table D-6-B-4 Reservoir Level at Time of Incident (for embankments after first filling)

Reservoir Level	Number of Incidents	Percentage of Incidents
Above normal operational level	11	15 %
At normal operational level *	52	70 %
Below normal operational level	11	15 %
TOTAL	**74**	

*Note: At normal operational level typically means within 2 feet of the normal annual high pool

The following observations from Table D-6-B-4 are of note.

- At Reclamation embankments that successfully completed initial reservoir filling and were in normal operational status, **approximately 85 percent of internal incidents**

were observed at reservoir levels at or below normal levels.

- Only 15 percent of internal erosion incidents after first filling were observed during higher than *normal* reservoir levels. At most Reclamation facilities, *"normal"* levels refer to the top of active conservation or joint use that the pool routinely reaches in most years. At a few facilities where the top of active is rarely reached, normal was assumed to be the typical upper levels reached in the past.

- Some of the incidents in this study clearly suggest that internal erosion had been slowly progressing for years or even decades. Reclamation believes that many internal erosion processes may take years to progress to the point of showing indications of distress or obvious particle transport. Thus, the reservoir level at the time of the incident may not reflect the key hydraulic head responsible for the initiation of internal erosion. **It is possible that internal erosion may occur during the several-week period of typical high reservoir levels at Reclamation facilities, and then stop for the season. In other words, internal erosion can be an episodic event that progresses to a limited extent only at normal high pools, year after year.**

Development of Base Rate Frequencies of Initiation of Internal Erosion

An estimate of the rate of initiation of internal erosion at a typical Reclamation embankment dam can be obtained by dividing the number of incidents and failures by the number of dam-years of operation. Since our current inventory of embankment dams contains all dams that have experienced a first filling (the youngest dam is 6 years old as of 2016), of specific interest are incidents at dams that have survived first filling. (For a dam safety evaluation of a new dam, these base rate frequencies reflecting the likelihood of initiation would not be appropriate and could be much higher. Furthermore, when considering the likelihood of initiation of internal erosion at flood levels, these base rate frequencies should be adjusted upward based on considerations such as potential for flaws in upper portion of the dam, amount of increased gradient expected, etc.)

When considering the potential for incidents after first filling, there seems to be a significant break at an age of 5 years as shown on Table D-6-B-5. Thus, the number of incidents after 5 years of operation will be considered in developing base rate frequencies for dams that have survived first filling. Furthermore, since Reclamation is particularly concerned with the risk of internal erosion at normal operation levels, another filter will be to consider only those incidents that occurred at or below normal reservoir levels. From this Reclamation incident database, there are 58 incidents that occurred at or below normal pool levels at dams that had at least 5 years of reservoir operation. These applicable incidents are summarized in Table D-6-B-5.

Table D-6-B-5 Number of Incidents of Internal Erosion at Reclamation Embankment Dams (after 5 years of operation and at or below normal pool level)

	Backward Piping	**Internal Migration**	**Scour**	**Suffusion/ Suffosion**	**TOTALS**
Embankment	0	1	5	1	**7**
Foundation	5	5	7	6	**23**
Emb into Fnd	0	0	5	0	**5**
Into/along Conduit	2	4	1	0	**7**
Into Drain	5	9	0	2	**16**
TOTALS	**12**	**19**	**18**	**9**	**58**

The total number of operational dam-years was obtained by identifying each of the 233 Reclamation embankments considered in this study, determining their present age (to year 2016), and summing all ages. In this manner, the number of dam-years of operation at Reclamation facilities was calculated to be 14,611. For all categories not involving the foundation only or into/along a conduit, this number of dam-years is considered to represent the operational history of the Reclamation embankments in the study.

However, adjustments are needed when considering internal erosion in the foundation and into/along conduits. As the incident database reflects, internal erosion through the foundation only is likely limited to embankments with partially penetrating cutoffs (with only 3 exceptions involving unusual bedrock/foundation conditions). Therefore, the appropriate operational history to use would be the number of dam-years associated with those Reclamation embankments featuring partial cutoffs – this value was computed to be 7,191 dam-years. Similarly, internal erosion into/along the conduits is only a viable scenario (with very rare exceptions) for those embankments that feature a penetrating conduit within the embankment. The number of dam-years associated with these specific embankments total 7,858.

Using these values of dam-years of operation and the number of incidents (under normal operations and after 5 years of operation) identified by the study, the following tables D-6-B-6 and -7 present the estimated historical rate at which erosion has initiated (and continued to progress to at least some degree).

Table D-6-B-6 Historical Rate of Initiation of Internal Erosion at Reclamation Embankments Based on Category

Internal erosion category	Estimated historical rate of erosion initiation*	
	Incidents/Failures with definitive particle transport	All incidents/failures
Embankment only	3.4×10^{-4}	4.8×10^{-4}
Foundation only	2.2×10^{-3}	3.2×10^{-3}
Embankment into foundation	1.4×10^{-4}	3.4×10^{-4}
Into/Along conduit	8.9×10^{-4}	8.9×10^{-4}
Into drain	1.1×10^{-3}	1.1×10^{-3}
TOTAL	**4.7×10^{-3}**	**6.0×10^{-3}**

***Note: See later discussion of whether these data include more than just "initiation"**

Table D-6-B-7 Historical Rate of Initiation of Internal Erosion at Reclamation Embankments Based on Mechanism

Internal erosion mechanism	Estimated historical rate of erosion initiation*	
	Incidents/Failures with definitive particle transport	All incidents/failures
Backward erosion piping	1.0×10^{-3}	1.3×10^{-3}
Internal migration	1.9×10^{-3}	1.9×10^{-3}
Scour	1.2×10^{-3}	1.8×10^{-3}
Suffusion/suffosion	6.2×10^{-4}	1.0×10^{-3}
TOTAL	**4.7×10^{-3}**	**6.0×10^{-3}**

***Note: See later discussion of whether these data include more than just "initiation"**

It is easy to question whether this review of past internal erosion incidents is a reasonable portrayal of performance at Reclamation embankments. Another way to consider this frequency question might be to note that it is not unusual, on average, to see 1 or maybe even 2 new "incidents" of unusual seepage or piezometric behavior, new sand boils, or new sinkholes each year within the Reclamation inventory of embankment dams. Assuming 1.5 incidents per year with 233 embankments equates to an annual frequency of 6.4×10^{-3}. This alternate approach to estimating a base rate frequency of the initiation of internal erosion happens to be similar to the summary value obtained from the incident study – although far from definitive, this does support a measure of confidence in the reasonableness of these frequency data.

Rather than directly use the values reflected in these tables, it is recognized that additional adjustments may better reflect the ranges of potential "best estimate" values given the variables and uncertainties involved with categorizing internal erosion events. One key uncertainty deals with whether the historical base rate of incidents portrayed above reflects more than just the "initiation" phase of the internal erosion process. In other words, initiation may have occurred in more Reclamation embankments than these catalogued because the process never "progressed" far enough to manifest symptoms like those detected or observed in the 97 cases.

It is difficult to estimate an additional number of dams where internal erosion may have initiated but did not continue or progress, and thus remained undetected. The original UNSW study of world-wide dams (Foster et al. 1998) assumed the number of "unreported" incidents of initiation was probably in the range of 2 to 10 times the number of reported incidents. Given Reclamation's reporting and documentation capabilities, it would seem likely that the lower portion of this range would be more applicable. In addition, there are some cases where the true source of the seepage and any signs of particle transport could not be conclusively tied to the reservoir. This is especially true in the cases of internal erosion through foundation and into drains, as other seepage sources (hillside, tailwater, etc.) may have provided the driving hydraulic head.

Furthermore, rather than specifying a single value, it appears to make more sense to suggest a range of **best estimate** values. The term "best estimate" is used since the true range of initiation of internal erosion probably spans several orders of magnitude. The lower end of the best estimate range is based on applying a multiplier of 1.25 to the observed 46 cases of definite particle transport, which assumes 25 percent more cases of definitive particle transport may have occurred and not noticed or reported. It seems rather unlikely that definitive evidence would not have been observed; thus a relatively small multiplier. (However, this multiplier was not applied to “foundation only” and “into drain” categories due to the belief that some incidents may have been attributed to seepage sources other than the reservoir.) The upper end of the best estimate range is based on a doubling of all 58 reported incidents, including the 12 that did not manifest any particle transport. Thus, the upper range values assume that only about 40 percent of all cases of definitive initiation of internal erosion have actually been observed and documented within Reclamation.

These adjusted values shown in the following tables are proposed as “starting points” or an empirical reference point for considering the probability of the initiation of internal erosion for Reclamation dams (or in an inventory of dams similar to Reclamation’s). It should be noted that this inventory includes approximately 200 dams constructed prior to the failure of Teton Dam without well designed filters. These tables are considered “summary” tables in that they provide a broad view of overall base rate frequencies.

Table D-6-B-8 Proposed Best Estimate Values of Annual Probabilities of Initiation of Internal Erosion by Category

Type of internal erosion	Range of initiation probability
Embankment only	$4x10^{-4}$ to $1x10^{-3}$
Foundation only*	$2x10^{-3}$ to $6x10^{-3}$
Embankment into foundation	$2x10^{-4}$ to $7x10^{-4}$
Into/Along conduit**	$1x10^{-3}$ to $2x10^{-3}$
Into drain*	$1x10^{-3}$ to $2x10^{-3}$

***Note: “Foundation only” and “Into drain” lower values were not adjusted by the 1.25 multiplier given the belief that some incidents may have been attributed to seepage sources other than the reservoir**

Table D-6-B-9 Proposed Best Estimate Values of Annual Probabilities of Initiation of Internal Erosion by Mechanism

Type of internal erosion	Range of initiation probability

Backward erosion piping	$1x10^{-3}$ to $3x10^{-3}$
Internal migration	$2x10^{-3}$ to $4x10^{-3}$
Scour	$2x10^{-3}$ to $4x10^{-3}$
Suffusion/suffosion	$8x10^{-4}$ to $2x10^{-3}$

Considerations for Usage of the Base Rate Frequency Tables

1. These ranges are considered to be "**best estimates**" – <u>not the reasonable low and reasonable high</u>. Higher or lower estimates of initiation probability may be appropriate if conditions at the dam being evaluated are better or worse than the "average" condition at a Reclamation dam. For example, dams with very low hydraulic gradients and minimal seepage may lead to lower estimates of initiation, while dams with appreciable seepage or a history of concerns may warrant higher estimates.

2. The incidents used to develop these values were limited to only Reclamation embankment dams, so use on embankments designed/constructed by others should consider how well those dams compare to Reclamation practices.

3. A total of 80 percent of the incidents featured soils with no or low plasticity (i.e., PI < 7). If postulated failure modes involve low plasticity soils, there is no need to consider higher estimates. However, lower initiation rates may be considered with more plastic soils.

4. The portrayed base rate frequencies have been developed for dams with more than 5 years of operational history and operating at normal reservoir levels. Thus, when evaluating new dams, consideration should be given to using somewhat higher values of initiation probability. Similarly, when estimating hydrologic risks, higher values of initiation probability would be expected when a dam is exposed to future higher reservoir levels.

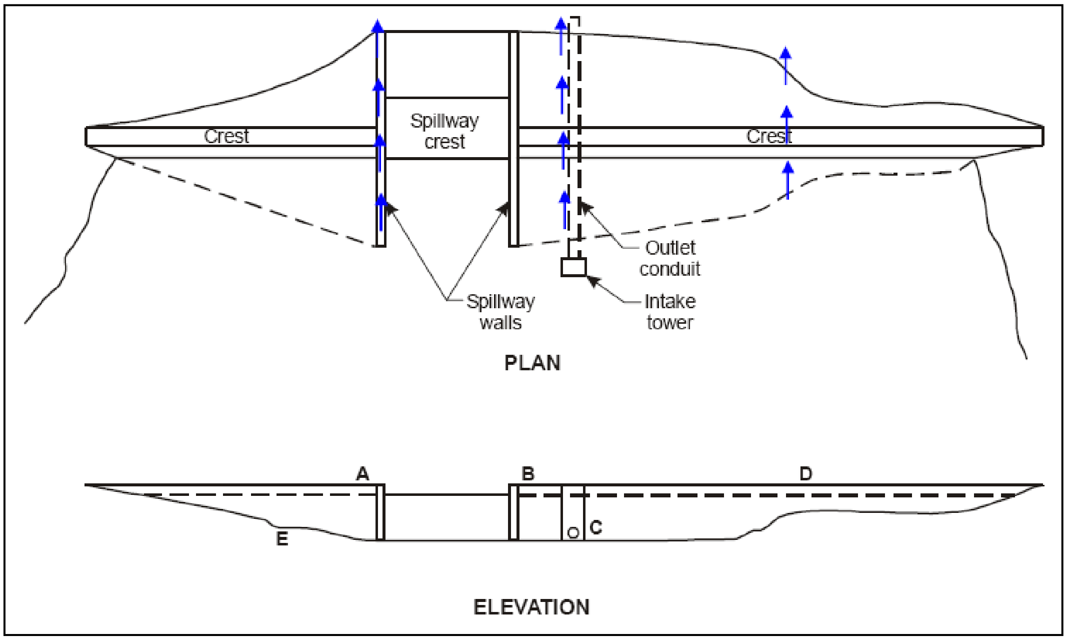
5. ***Simply referring to the tabulated best estimates from Reclamation's history of incidents is not sufficient in evaluating the probability for erosion to initiate.*** Instead, site conditions must be considered in order to determine whether there are features, conditions, or behaviors present at a given site that will influence the potential for erosion to initiate. Comprehensive tables have been developed that offer a number of considerations that would affect the likelihood of initiation at a given site. There are separate tables for each category of internal erosion. Any estimate of initiation should consider the factors in the 11x17 tables in Appendix D-6-J.

Appendix D-6-C: Concentrated Leak Erosion

Given a crack or gap (i.e., flaw) exists, the initiation of concentrated leak erosion depends on the depth or location of cracking relative to the headwater level and the forces imposed on the sides of the crack by water flowing through it. The resistance to initiation of concentrated leak erosion is characterized by the critical shear stress (see Chapter D-1 Erosion of Rock and Soil). To help assess the likelihood of initiation of concentrated leak erosion in a crack or gap, the hydraulic shear stress in the crack for the headwater level under consideration (τ) can be compared to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c) at the degree of saturation of the soil on the sides of the crack. Further details are provided in this appendix.

Estimation of crack widths and depths involves a lot of uncertainty. ICOLD (2013) provides some examples of likely crack depths and widths due to cross-valley differential settlement or differential settlement in the foundation. These estimates are based primarily on the methods described in Fell et al. (2008), which includes methods for transverse cracking due to differential settlement, frost action, and desiccation, as well as hydraulic fracture. If a crack forms during construction, it may be masked by lifts placed near the crest after most of the deformation is complete, which may not propagate a crack upward, at least not to the same openness.

Common Situations where Concentrated Leaks May Occur



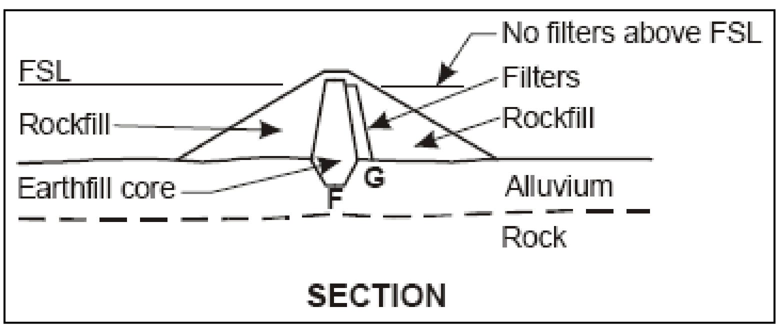


Figure D-6-C-1 Potential Failure Paths (Fell et al. 2008)

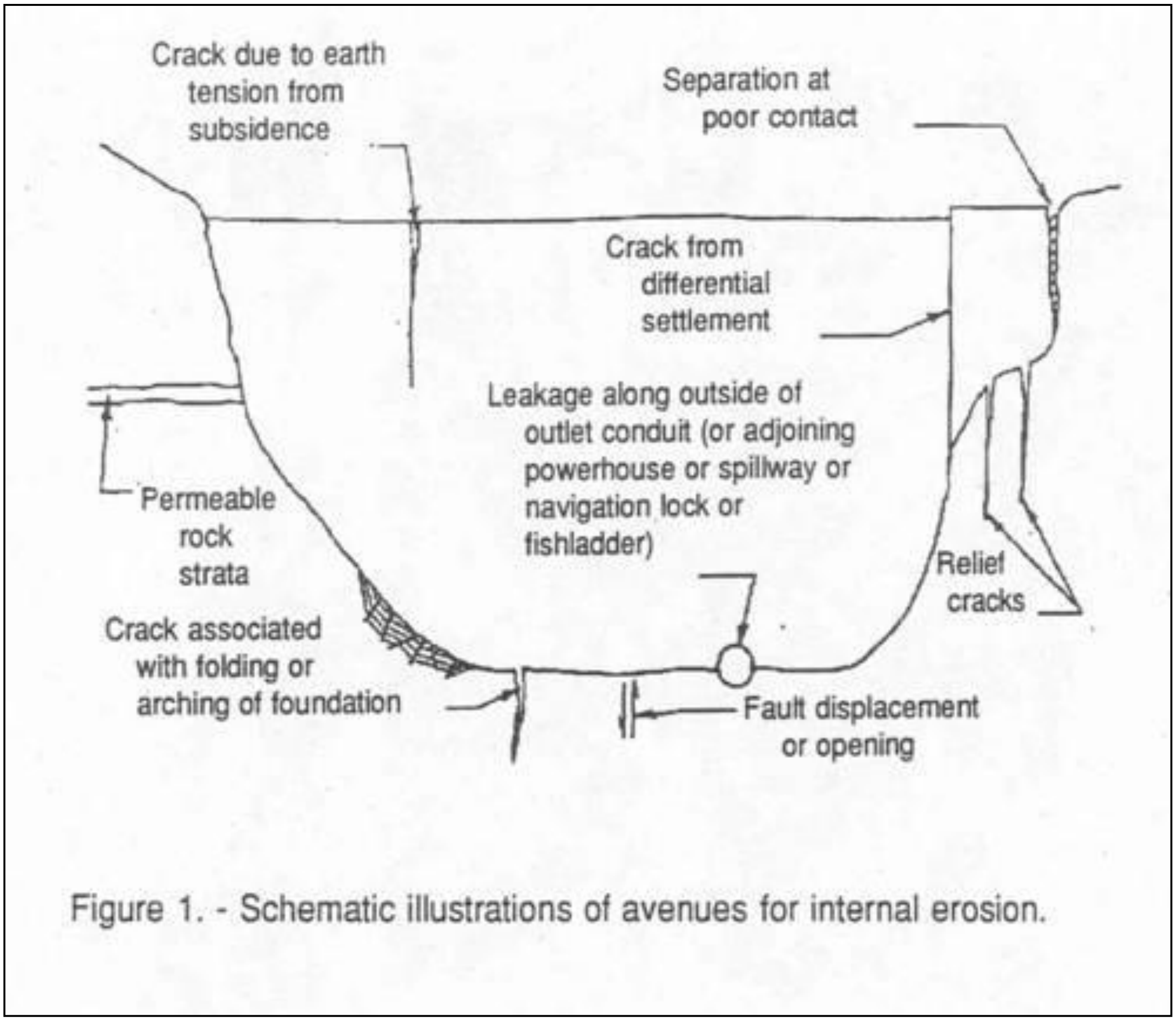


Figure D-6-C-2 Potential Failure Paths (Unknown Source)

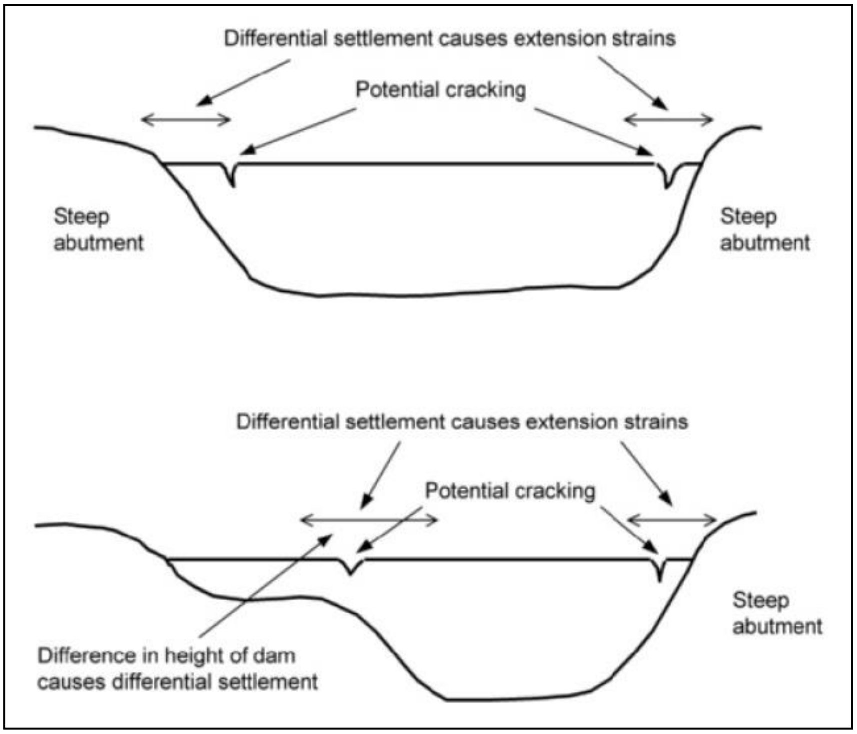


Figure D-6-C-3 Cracking and Hydraulic Fracture due toCross-Valley Differential Settlement of the Core (Fell et al. 2014)

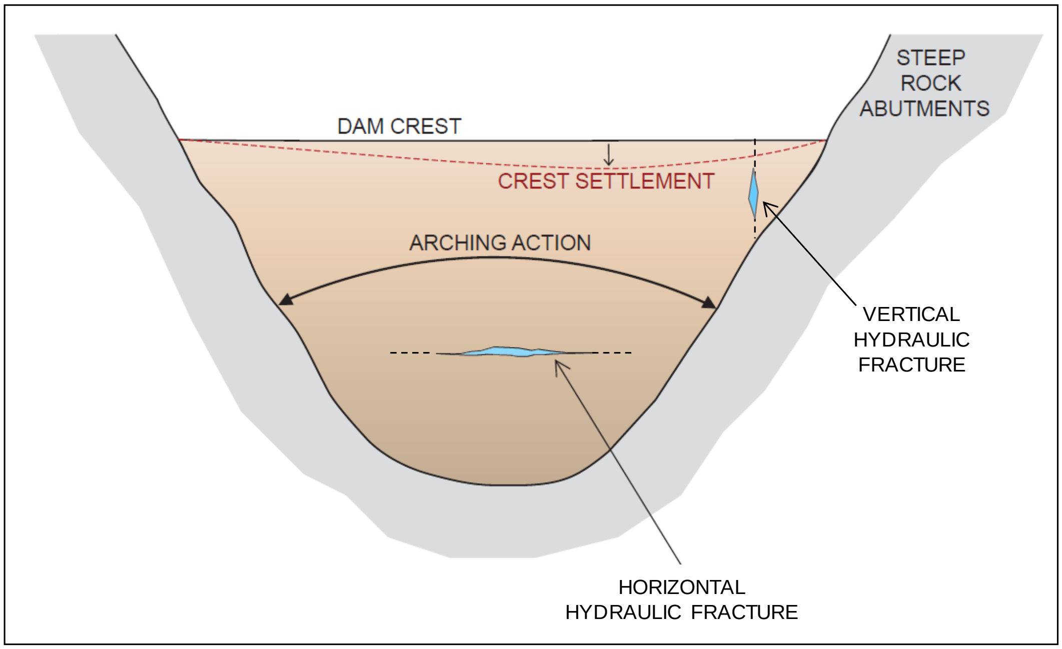


Figure D-6-C-4 Cracking and Hydraulic Fracture due to Cross-Valley Arching and Steep Abutment Slopes (Courtesy of Mark Foster)

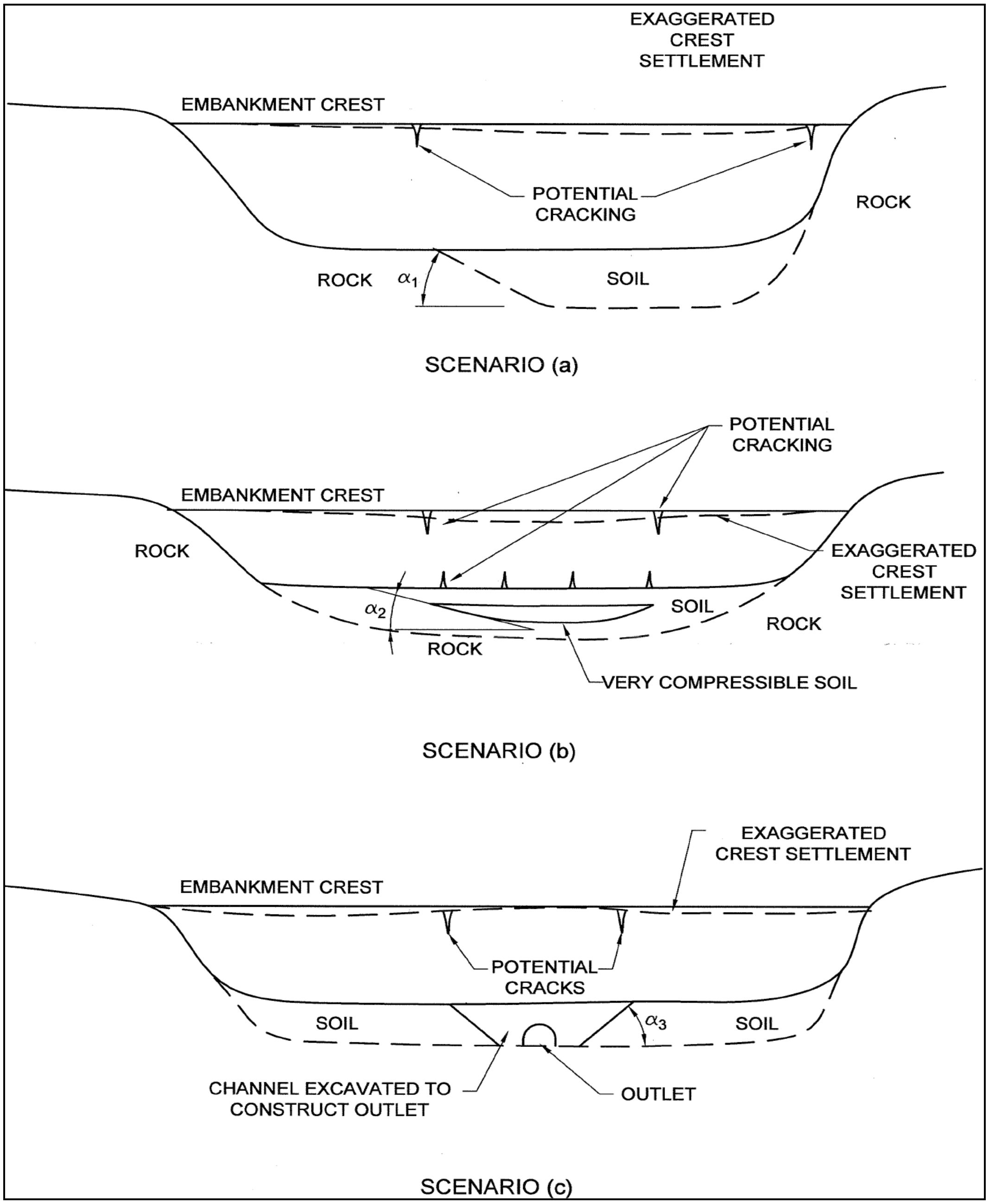


Figure D-6-C-5 Cracking and Hydraulic Fracture due to Differential Settlement in the Foundation (Fell et al. 2008)

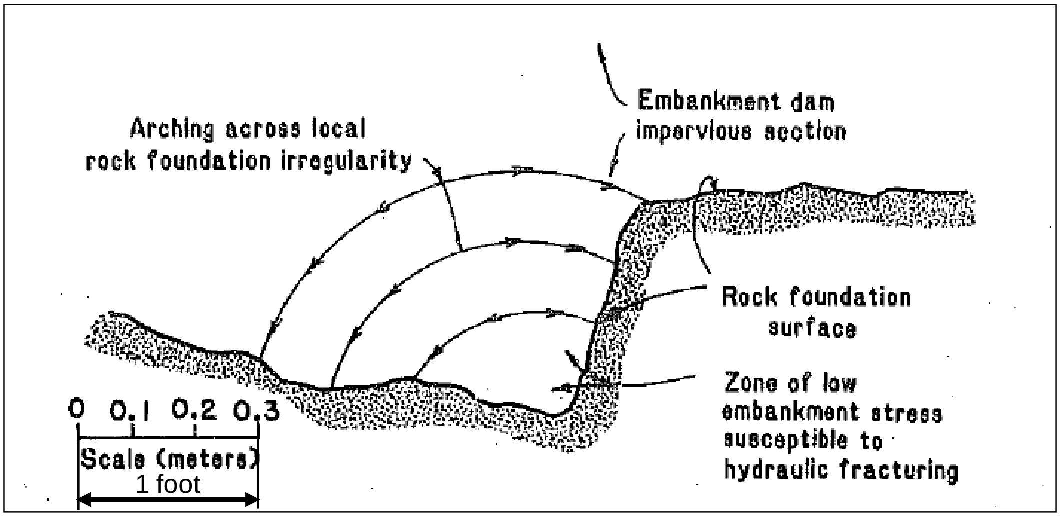


Figure 6

Figure D-6-C-6. Cracking and Hydraulic Fracture due to Small-Scale Irregularities in the Foundation Profile (Sherard 1985b)

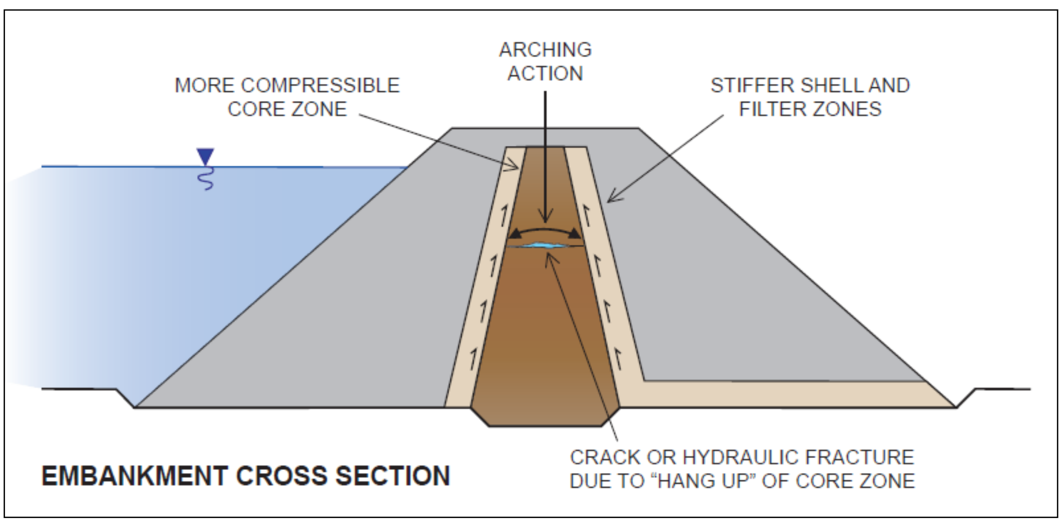


Figure 7

Figure D-6-C-7. Cracking and Hydraulic Fracture due to Arching of Core onto Embankment Shells (Courtesy of Mark Foster)

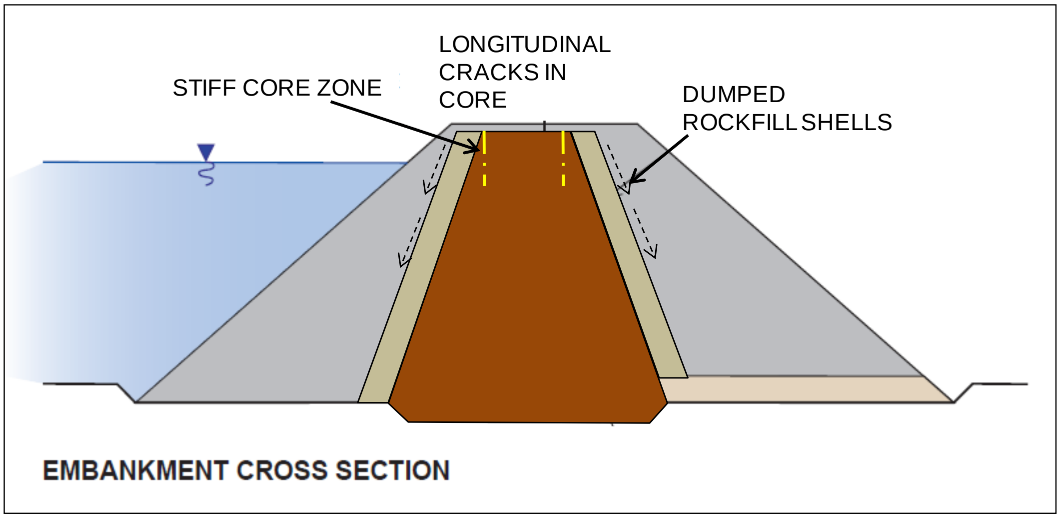


Figure 8

Figure D-6-C-8. Cracking due to Cross-Sectional Settlement (Differential Settlement of Shell Zones) (Courtesy of Mark Foster)

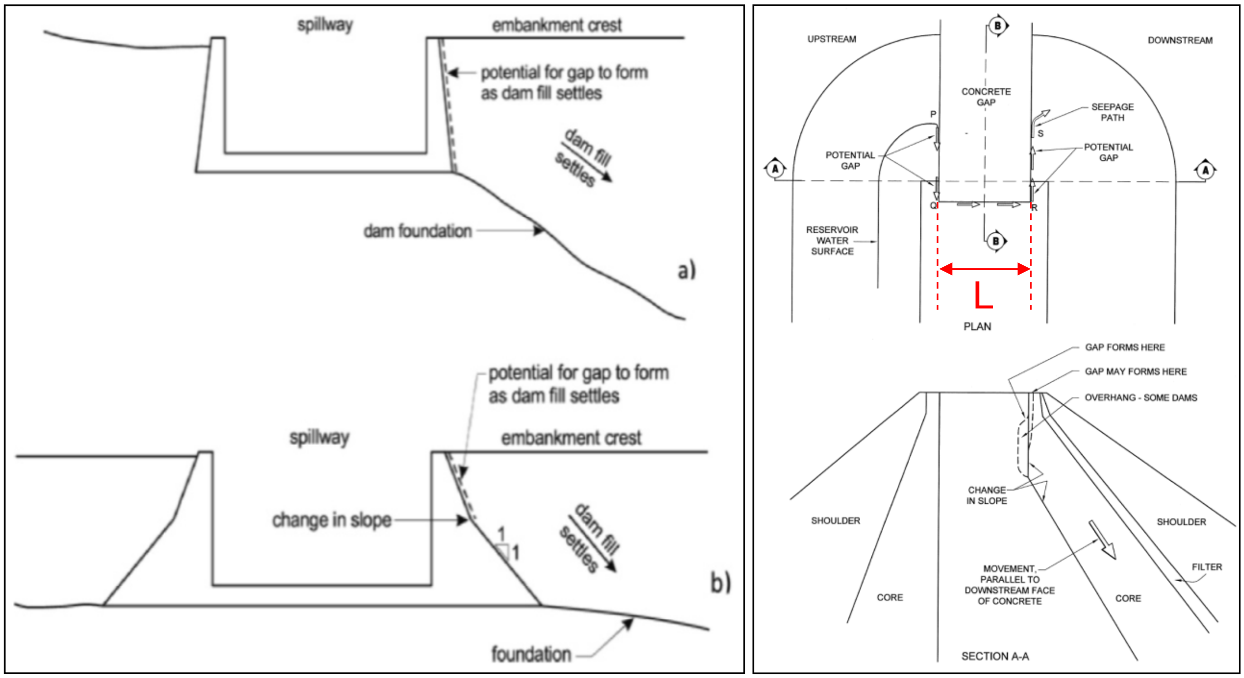


Figure 9

Figure D-6-C-9. Crack or Gap adjacent to Spillway or Abutment Walls and Embankment-Concrete Interfaces(Fell et al. 2008)

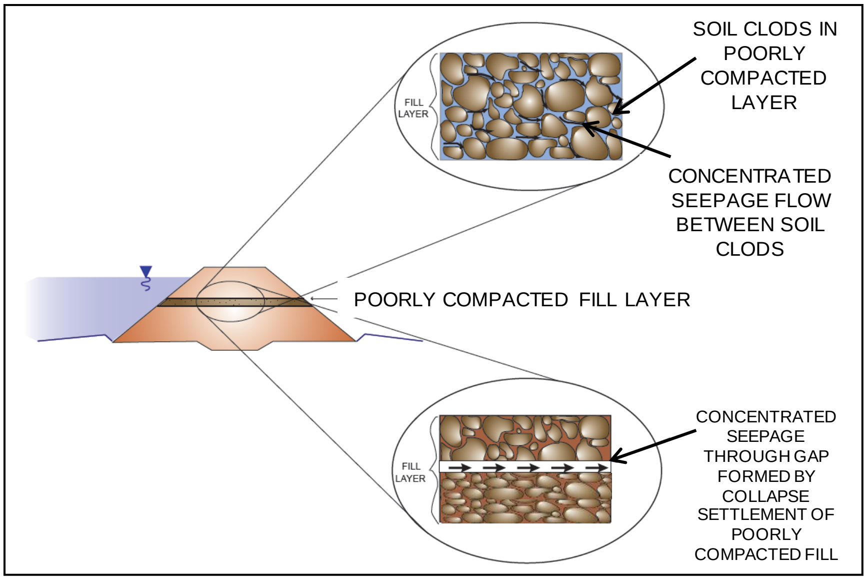


Figure D-6-C-10 Crack, Hydraulic Fracture, or Openings in Poorly Compacted and/or Segregated Layers (Courtesy of Mark Foster)

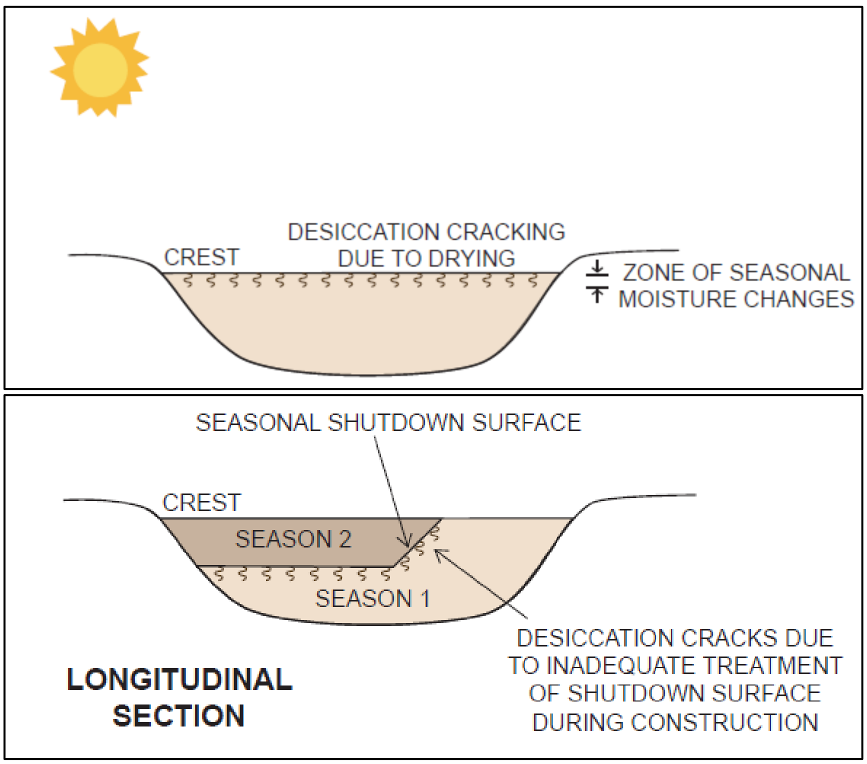


Figure D-6-C-11 Cracking due to Desiccation (Courtesy of Mark Foster)

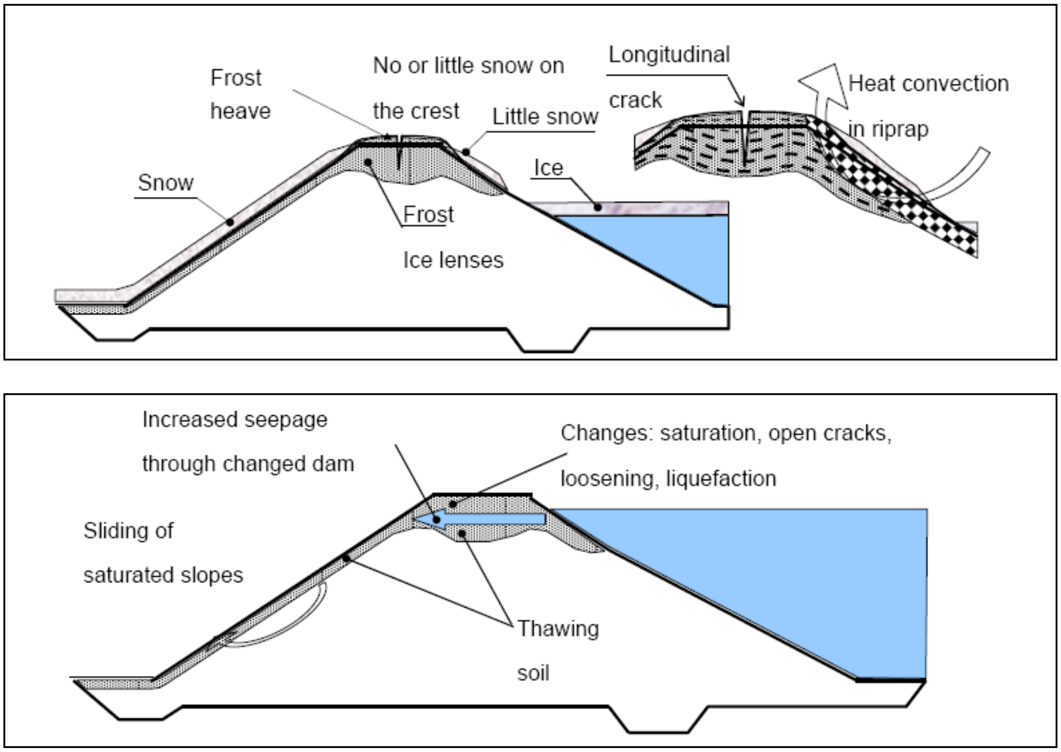


Figure D-6-C-12 Cracking or High-Permeability Layers due to Freezing (Vuola et al. 2007)

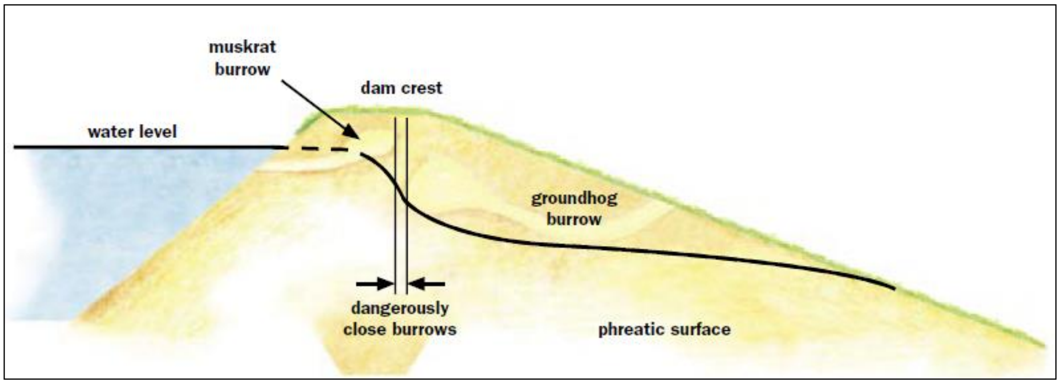


Figure D-6-C-13 Effects of Animal Burrows (FEMA 2005)

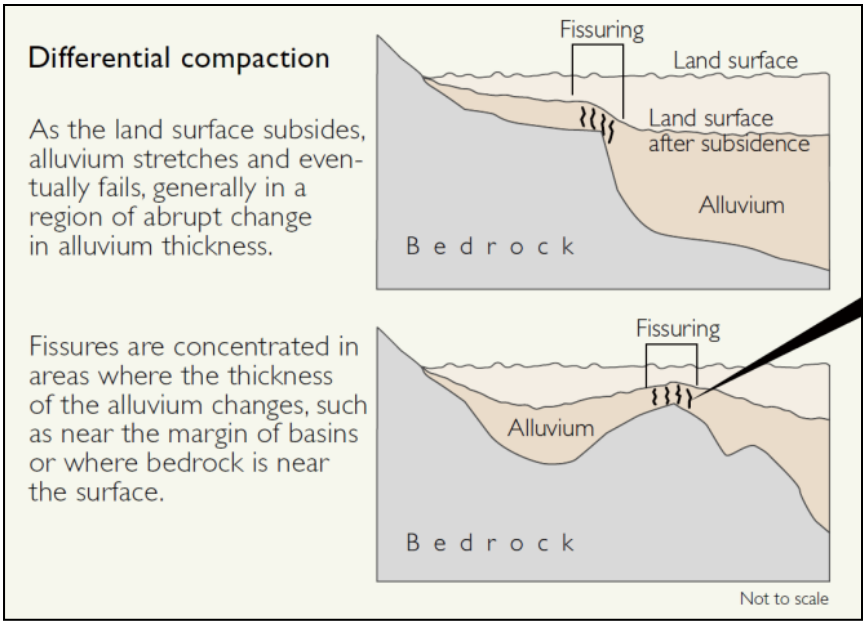


Figure D-6-C-14 Cracking caused by Earth Fissures due to Subsidence (Galloway et al. 1999)

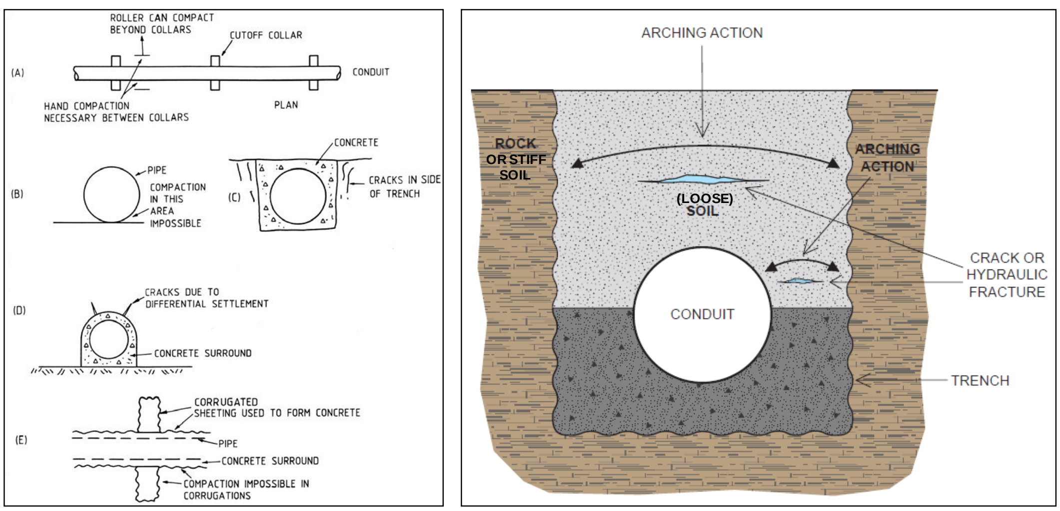


Figure D-6-C-15 Some Causes of Piping Failures around Conduits (Fell et al. 2008)

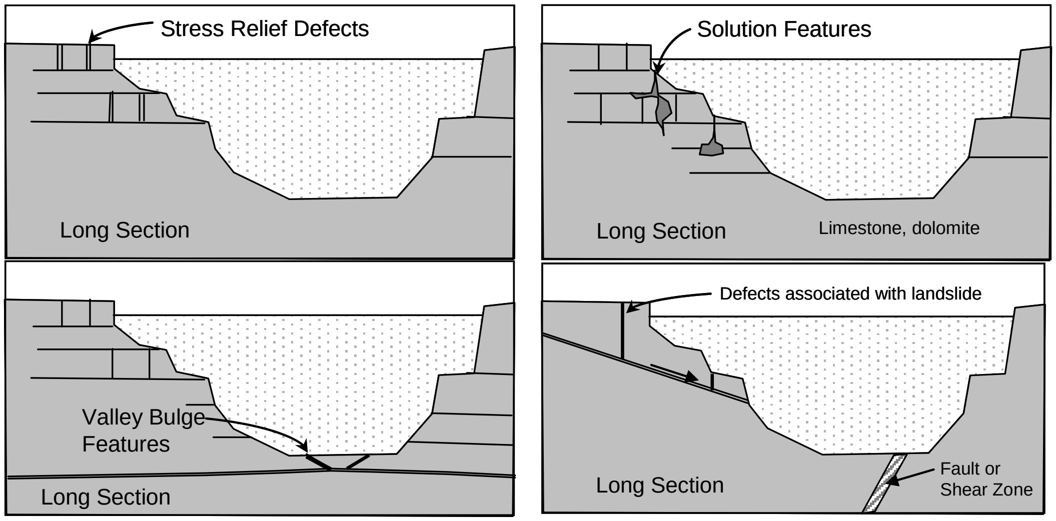


Figure D-6-C-16 Defects in Rock Foundations due to Geologic Processes (Fell et al. 2008)

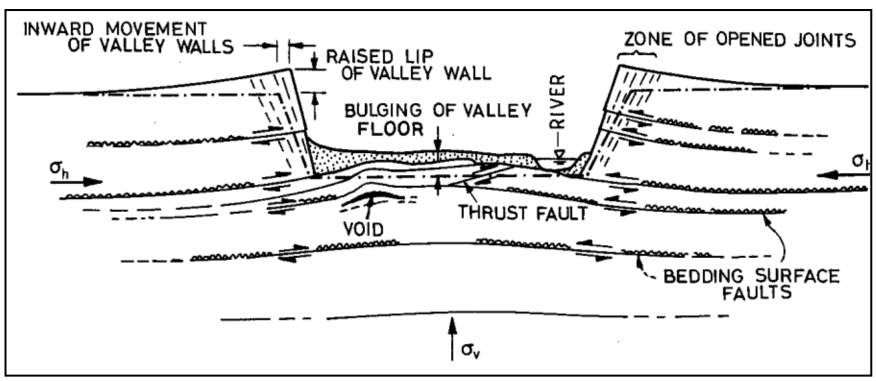


Figure D-6-C-17 Valley Rebound and Stress Relief Effects in Valleys in Sedimentary and Other Horizontally Bedded Rocks (Fell et al. 2008)

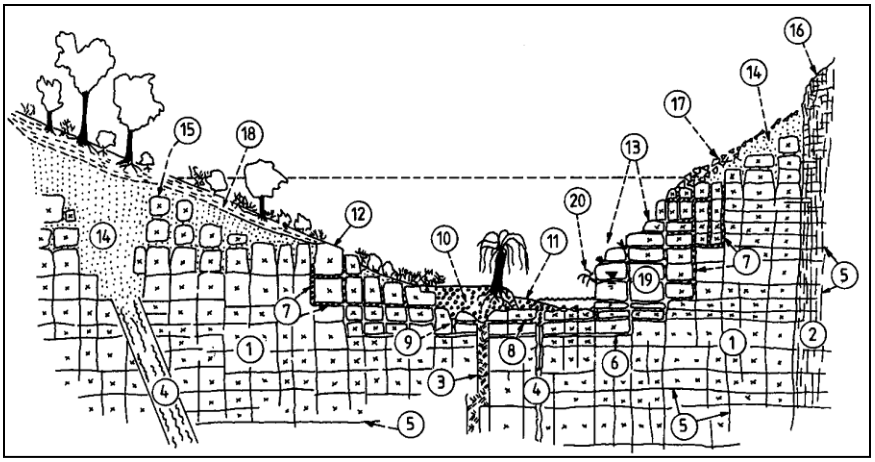


Figure D-6-C-18 Features in Valleys Formed in Strong Jointed Rocks (Fell et al. 2008)

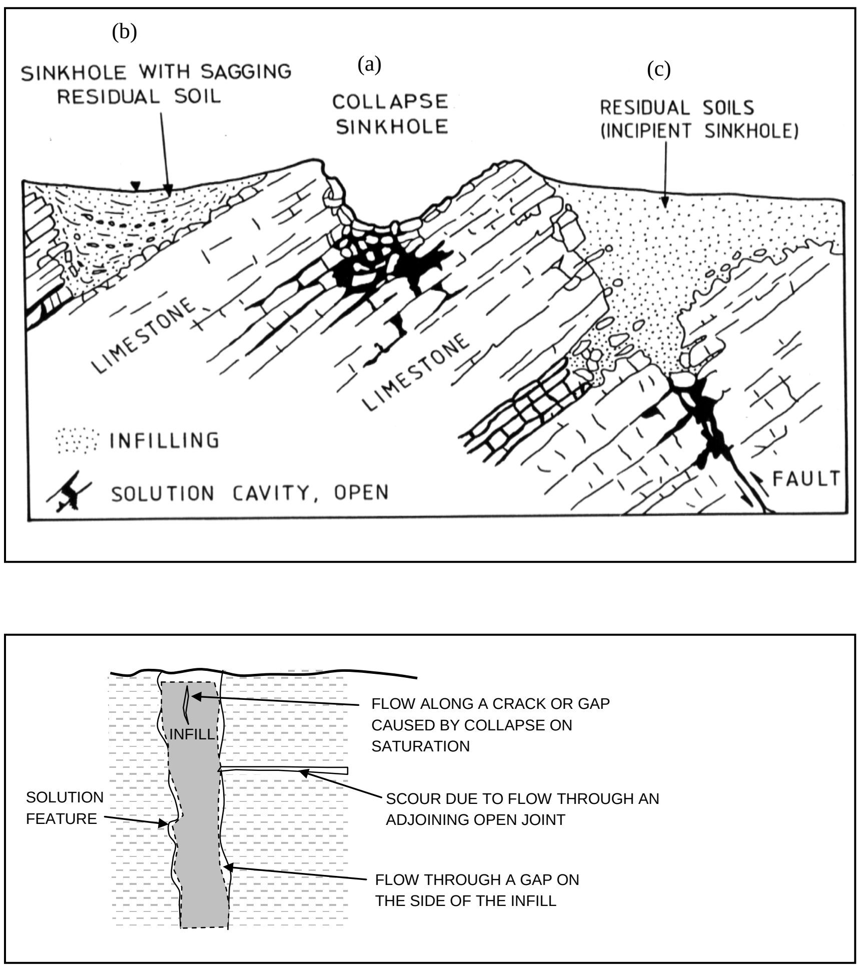


Figure D-6-C-19 Defects in Rock Foundations (Fell et al. 2008)

Hydraulic Shear Stress

The hydraulic shear stress in a crack or pipe for the headwater level under consideration is based on the geometry of the embankment core, the assumed pipe or crack dimensions, and the location of the pipe or crack relative to the headwater level so that the flow gradient and velocity can be determined. According to Wan and Fell (2004b), the hydraulic shear stress can be estimated by the following equation:

$$\tau = \rho_w g \left(\frac{\Delta H}{L}\right)\left(\frac{A}{P_w}\right)$$

Equation D-6-C-1

where ρ_w = density of water; g = acceleration due to gravity; ΔH = hydraulic head difference; L = length of pipe or crack over which the hydraulic head difference occurs; A = cross-sectional area of pipe or crack; and P_w = wetted perimeter of pipe or crack. Since the unit weight of water, $\gamma_w = \rho_w g$ and the hydraulic gradient, $i = \Delta H/L$, then the expression can be simplified to the following:

$$\tau = \gamma_w i \left(\frac{A}{P_w}\right)$$

Equation D-6-C-2

Using this basic equation and the estimated geometry of the pipe or crack, the following approximations can be derived for the hydraulic shear stress.

The assumptions for the estimation of the hydraulic shear stress are:

- Linear head loss from upstream to downstream
- Steady uniform flow along the pipe or crack
- Zero pressure head at the downstream end
- Uniform frictional resistance along the surface of the pipe or crack
- Driving force = frictional resistance

Cylindrical Pipe

For cylindrical pipe, the equation for hydraulic shear stress is the following:

$$\tau = \frac{\rho_w g (H/L)(\pi D^2/4)}{\pi D} = \frac{\rho_w g H D}{4L}$$ **Equation D-6-C-3**

where τ = hydraulic shear stress; ρ_w = density of water; g = acceleration due to gravity; H = hydraulic head at upstream end; L = length of pipe at mid-depth of headwater level under consideration; and D = diameter of pipe.

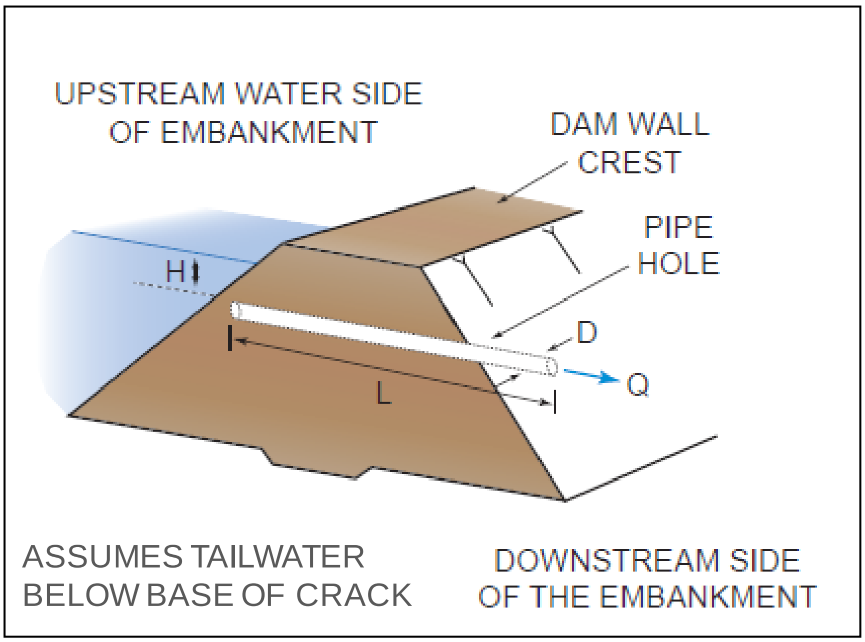


Figure D-6-C-20 Cylindrical Pipe Geometry (Fell et al. 2014)

Since $\gamma_w = \rho_w g$ and $i = H/L$, then

$$\tau = \gamma_w i \left(\frac{D}{4}\right)$$ **Equation D-6-C-4**

Vertical Parallel-Sided Transverse Crack

For vertical parallel-sided transverse crack, the equation for hydraulic shear stress is the following:

$$\tau = \frac{\rho_w g(H/L)(HW)}{2(H+W)} = \frac{\rho_w g H^2 W}{2(H+W)L}$$ **Equation D-6-C-5**

where τ = hydraulic shear stress; ρ_w = density of water; g = acceleration due to gravity; H = hydraulic head at upstream end; L = length of crack at mid-depth of headwater level under consideration; and W = width of crack.

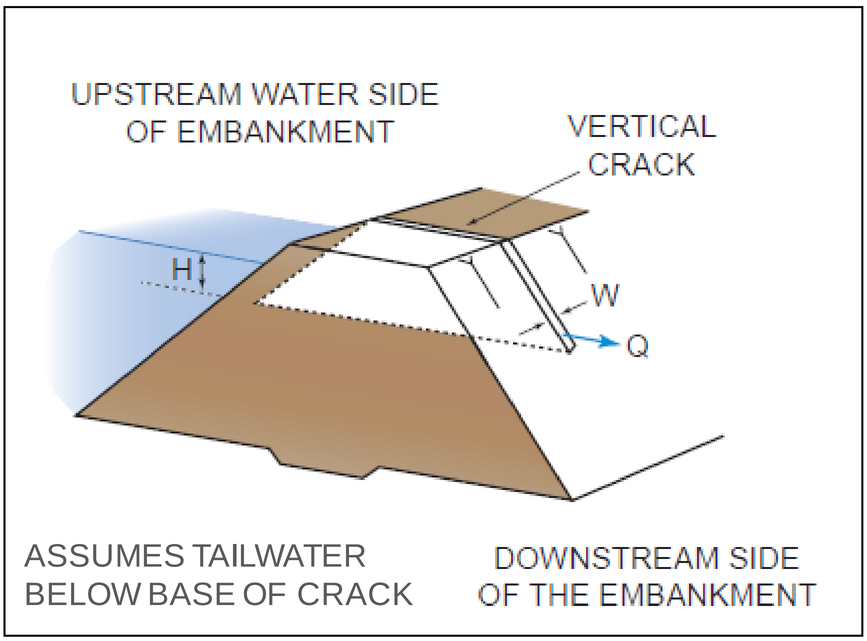


Figure D-6-C-21 Vertical Parallel-Sided Transverse Crack Geometry (Fell et al. 2014)

Since H + W ≈ H (because H >> W), $\gamma_w = \rho_w g$ and i = H/L, then the equation can be simplified to the following:

$$\tau \approx \gamma_w i \left(\frac{W}{2}\right)$$ **Equation D-6-C-6**

Vertical Uniformly Tapered Transverse Crack

For vertical uniformly tapered transverse crack, the equation for hydraulic shear stress is the following:

$$\tau = \frac{\rho_w g(H/L)(HW_H/2)}{2(H^2 + W_H^2/4)^{0.5} + W_H} \qquad \textbf{Equation D-6-C-7}$$

where τ = hydraulic shear stress; ρ_w = density of water; g = acceleration due to gravity; H = hydraulic head at upstream end; L = length of crack at mid-depth of headwater level under consideration; and W_H = width of crack at H.

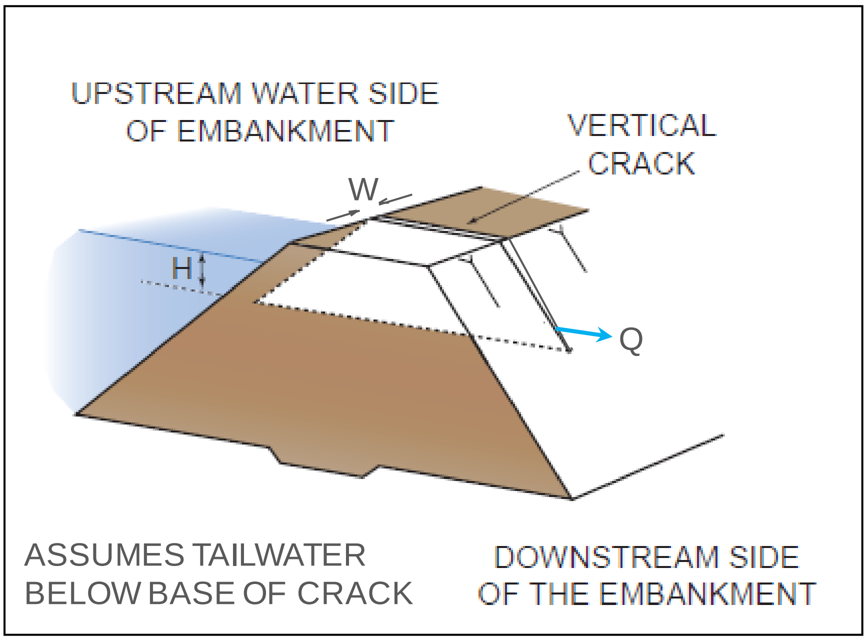


Figure D-6-C-22 Uniformly Tapered Transverse Crack Geometry (adapted from Fell et al. 2014)

Since $2(H^2 + W_H^2/4)^{0.5} + W_H \approx 2H$ (because $H >> W_H$), $\gamma_w = \rho_w g$ and $i = H/L$, then the equation can be simplified to the following:

$$\tau \approx \gamma_w i \left(\frac{W_H}{4}\right) \qquad \textbf{Equation D-6-C-8}$$

Critical Crack Width

The above approximate relationships can also be used in a reverse manner to estimate the critical continuous pipe diameter or crack width for initiation, which the team can then use as a more likely or less likely factor in assessing the likelihood of a flaw and initiation of concentrated leak erosion:

$D_c = \frac{4\tau_c}{i\gamma_w}$ for cylindrical pipe

$W_c = \frac{2\tau_c}{i\gamma_w}$ for vertical parallel-sided transverse crack

$W_c = \left(\frac{4\tau_c}{i\gamma_w}\right)\left(\frac{D}{H}\right)$ for vertical uniformly tapered transverse crack

Initiation

The critical shear stress (τ_c) can be compared to the estimated hydraulic shear stress for the headwater level under consideration (τ) to help assess the likelihood of initiation of concentrated leak erosion. The factor of safety can be estimated as:

$$FS = \frac{\tau_c}{\tau}$$ **Equation D-6-C-9**

Sensitivity or uncertainty analysis is recommended. In addition to a best estimate, a range of values should be considered from a reasonable low estimate to a reasonable high estimate. Probability distributions can also be assigned for the crack geometry and critical shear stress to be used in a Monte Carlo simulation to assess the probability of a factor of safety against initiation of concentrated leak erosion less than one.

Exceeding the limit-state condition simply provides an indication of the likelihood for concentrated leak erosion to initiate and progress. Analytical results should be used to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

An example of portrayal of analytical results with sensitivity analysis is shown in Figure D-6-B-23. In this example, a best estimate for critical shear stress was estimated by a risk team during an elicitation, along with reasonable low and reasonable high estimates. The hydraulic shear stress was then estimated for a range of pipe diameters and headwater levels. Based upon the estimated pipe diameter or range of pipe diameters for the flaw, this figure can be used to help develop a list or more likely and less likely factors for initiation of concentrated leak erosion as a function of headwater level.

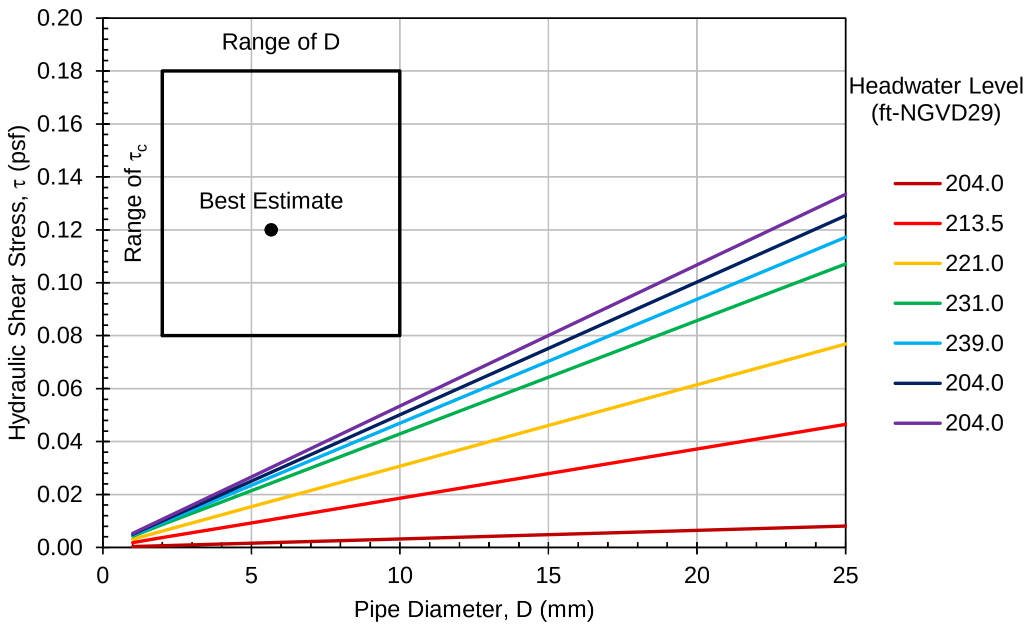


Figure D-6-C-23 Sample Portrayal of Analytical Results for Initiation of Concentrated Leak Erosion

Appendix D-6-D: Soil Contact Erosion

Geometric Condition (Screening-Level Assessment)

Fine soil layers that do not satisfy geometric criteria for filtration are not susceptible to soil contact erosion. Assessing the susceptibility to soil contact erosion for any risk assessment starts with a review of the particle-size distribution (i.e., geometric condition). Several researchers have proposed expressions for the geometric and hydraulic conditions, which are summarized in this appendix. Those geometric condition are very similar to the "no erosion" condition ($D_{15}/d_{85} < 9$) of Foster and Fell (1999, 2001). Modern filter design criteria can be used as an initial screening and must be used for the design of a new filter. For assessment of existing dams and levees, the Foster and Fell criteria (1999, 2001) can be used to assess filters (coarse layers) that do not satisfy modern filter design criteria (i.e., "no erosion" condition).

Experimental results of soil contact erosion for non-plastic soils for the geometric and hydraulic conditions for the detachment and transport of particles resulted in the domains shown in Table D-6-D-1. For the "geometric domain" where D_{15}/d_{85} is less than the thresholds in the third column, initiation of soil contact erosion is very unlikely to occur because there is geometrical filtration regardless of the hydraulic loading.

Table D-6-D-1 Domain of Geometric and Hydraulic Influence for Non-Plastic Soils(Bonelli 2013)

<table>
<tr><td></td><td colspan="5">Grading ratio D_{15}/d_{85} →</td></tr>
<tr><td>Brauns (1985)
soil with n=0.4</td><td rowspan="3">Geometrical condition</td><td>7.5</td><td rowspan="3">Geometrical and Hydraulic condition</td><td>25</td><td rowspan="3">Hydraulic condition</td></tr>
<tr><td>Wörman (1992)
soil with $D_{15}=0.88D_H$</td><td></td><td>14.6</td></tr>
<tr><td>Den Adel (1994)
soil with $d_{85}=d_{50}/0.9$</td><td>8.1</td><td>11.7</td></tr>
</table>

Hydraulic Condition

The critical gradient in the coarse layer can vary significantly depending on its permeability. However, the "critical" Darcy velocity for initiation of soil contact erosion does not significantly depend on its permeability and is only related to the fine soil's resistance to erosion. Therefore, the Darcy velocity is often a good indicator of the hydraulic loading and compared to the critical velocity for initiation of soil contact erosion. The critical velocity can be compared to the estimated Darcy velocity for the headwater level under consideration to help assess the likelihood of initiation and progression of soil contact erosion.

The hydraulic condition for soil contact erosion depends of the configuration of the fine and coarse layers. The influence of the coarse layer on the initiation of soil contact erosion can be neglected if D_{15}/d_{85} is greater than the values listed in the fifth column of Table D-6-D-1 for the "hydraulic condition" domain. For those situations, the hydraulic loading condition controls, and there is no filtration effect. In the "geometrical and hydraulic condition" domain, the critical velocity is also a function of the coarse soil grading, and the hydraulic loading to initiate soil contact erosion is higher than the "hydraulic condition" domain.

Braun (1985) proposed an expression for critical velocity which provides a good approximation for sand below gravel:

Equation D-6-D-1

$$U_{crit}\ \text{(m/s)} = 0.65 n_F \sqrt{\left(\frac{\rho_s - p_w}{p_w}\right) g d_{50}} = 0.65 n_F \sqrt{(G_s - 1) g d_{50}}$$

where n_F = porosity of the coarse soil (gravel); ρ_s = density of the base soil (sand) particles (kg/m^3); ρ_w = density of water (1,000 kg/m^3); G_s = specific gravity of the sand particles; g = acceleration of gravity (9.81 m/s^2); and d_{50} = mean grain size of the base soil (sand).

Guidoux et al. (2010) measured critical velocities and critical hydraulic gradients for various base soils and recommended using the effective grain diameter (d_H) of Koženy (1953) instead of d_{50} for a more representative particle-size description for the base soil to predict the critical velocity:

$$d_H = \left(\sum_{j=1}^{m} \frac{F_j}{d_j}\right)^{-1}$$

Equation D-6-D-2

where d_j (mm) = particle-size of the fraction j of the base soil gradation curve; and F_j (-) = mass fraction of the fraction j. For a well-graded soil, $d_H \approx d_{50}$. Their expression for critical velocity can be used for sands, silts, or sand/clay mixtures below gravel:

$$U_{crit}\ \text{(m/s)} = 0.65 n_F \sqrt{\left(\frac{\rho_s - p_w}{p_w}\right) g d_H \left(1 + \frac{\beta}{d_H^2}\right)}$$
$$= 0.65 n_F \sqrt{(G_s - 1) g d_F \left(1 + \frac{\beta}{d_H^2}\right)}$$

Equation D-6-D-3

where n_F = porosity of the coarse soil (gravel); ρ_s = density of the base soil particles (kg/m^3); ρ_w = density of water (1,000 kg/m^3); g = acceleration of gravity (9.81 m/s^2); G_s = specific gravity of the sand particles; d_H = effective grain diameter of the base soil; and β = empirical coefficient. Several parameters influence the coefficient β, which was estimated by Guidoux et al. (2010) by fitting the above equation to the experimental data and assuming it did not vary among the tested soils. The best fit obtained for β was 5.3E-09 m^2.

The relationships for critical velocity for both methods are shown in Figure D-6-D-1. Both methods give the same results for sand below gravel. Since the D_{50} of sand can be readily assessed from the gradation curves, the Brauns (1985) method is the simplest to use and provides a good approximation for sand below gravel. For other “base” soils, the Guidoux et al. method must be used.

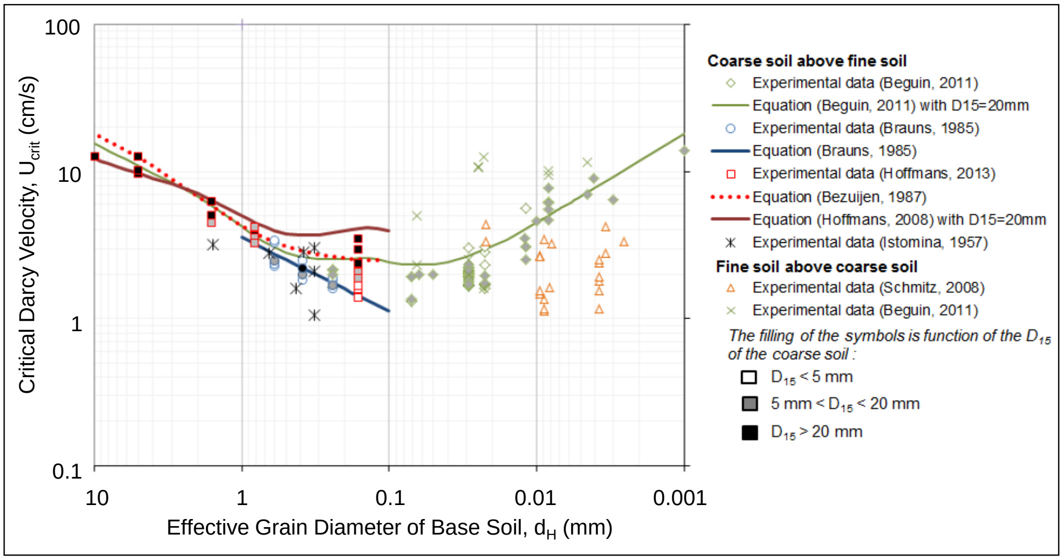


Figure D-6-D-1 Critical Velocity for Initiation and Progression of Soil Contact Erosion (ICOLD 2015)

Schmitz (2007) conducted testing for erosion of silt layers above coarse layers. In contrast to the configuration of fine soil below coarse soil, he noticed an influence of the confining stress on the critical velocity. For higher vertical stresses on the sample, he measured higher critical velocities. Generally, the critical velocities measured were of the same order of magnitude as the reverse configuration, between 1 and 10 cm/s but lower than the critical velocities proposed by Guidoux et al. (2010).

Initiation

The critical velocity can be compared to the estimated Darcy velocity for the headwater level under consideration to help assess the likelihood of initiation and progression of soil contact erosion. The factor of safety can be estimated as:

$$FS = \frac{U_{crit}}{k_h i}$$

Equation D-6-D-4

where k_h = hydraulic conductivity (horizontal) of the coarse layer; and i = seepage gradient for the reservoir level under consideration. Note this is Darcy velocity and does not need adjustment for porosity.

Sensitivity or uncertainty analysis is recommended. In addition to a best estimate, a range of values should be considered from a reasonable low estimate to a reasonable high estimate. Probability distributions can also be assigned for the mean grain size of the base soil (sand), effective grain diameter (d_H) of the base soil, and hydraulic conductivity (horizontal) of gravel to be used in a Monte Carlo simulation to assess the probability of a factor of safety against initiation of soil contact erosion less than one.

Exceeding the limit-state condition simply provides an indication of the likelihood for soil contact erosion to initiate and progress. Analytical results should be used to help to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

An example of portrayal of analytical results with sensitivity analysis is shown in Figure D-6-D-2. In this example, a range of hydraulic conductivity and effective grain diameter of the base soil were estimated by a risk team during an elicitation. Based on the estimated Darcy velocities, this figure can be used to help develop a list or more likely and less likely factors for initiation of and progression of soil contact erosion as a function of reservoir level.

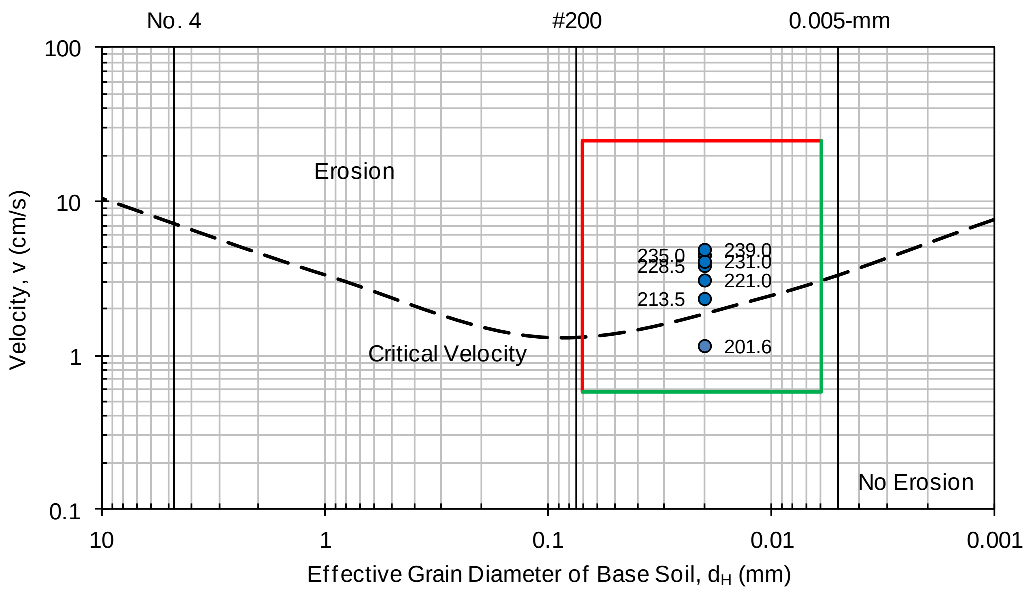


Figure D-6-D-2 Example Portrayal of Analytical Results for Initiation of Soil Contact Erosion

Appendix D-6-E: Critical Gradients for Evaluation of Backward Erosion Piping

Terzaghi et al. (1996) showed that backward erosion piping will initiate when a “heave” or zero effective stress condition occurs in sands subject to upward through-seepage. The basis for design guidance is to prevent the uplift or blowout condition, and thus initiation and progression of backward erosion piping. Based on experience with Mississippi River flooding, USACE developed an analytical procedure for assessing levee underseepage and vertical exit gradients commonly known as “blanket theory” for seven scenarios (with and without confining layers) which are described in EM 1110-2-1913 (USACE 2000). Flow nets and two-dimensional finite element modeling (e.g., SEEP/W) are two commonly used techniques to estimate gradients.

To sustain piping, the seepage flow must be maintained at or above the critical gradient, and a mechanical condition is necessary to sustain a continuous roof for the developing pipe either by the embankment or a confining layer. Test results from studies by Weijers and Sellmeijer (1993), Schmertmann (2000), Sellmeijer et al. (2011) have shown that backward erosion can progress at global gradients of 40 to 60 percent of the critical gradients to cause backward erosion to initiate, especially for fairly uniform, fine to medium sands where critical global gradients can be as low as 0.02.

To help assess the likelihood of the hydraulic condition for progression of backward erosion piping, the global or horizontal gradient in the foundation can be compared to the critical gradient for progression of a pipe. Methods to evaluate the critical gradient for progression of a pipe include line-of-creep methods (Bligh 1910 and Lane 1935), Sellmeijer’s piping rule (1993, 2011), and Schmertmann’s methodology (2000, 2016). ***Multiple methods are suggested to help inform judgment. The correct application of these methods requires an understanding of the context from which each method was developed.*** Robbins and van Beek (2015) provide a more detailed review of the background, advantages, and disadvantages of each method and the various laboratory test conditions (e.g., density, exit configuration, soil characteristics, and scale effects) that significantly impact the findings. For example, the Sellmeijer and Schmertmann “average gradient” methods can only be used for situations that have a purely two-dimensional seepage regime (i.e., only applicable to situations that have uniform boundary conditions parallel to the embankment centerline such as an exposed ditch or no confining layer). Some methods may not apply to the materials under consideration. For example, Sellmeijer's piping rule is only applicable within the range of soils tested. ***For soils beyond the suggested ranges and differing exit configurations, the methods are not necessarily applicable, and the actual critical gradients may be quite different than what is estimated.***

All other parameters remaining the same, the likelihood of backward erosion piping is:

- Decreased by increasing particle size
- Decreased by increased coefficient of uniformity
- Decreased by increasing relative density
- Decreased by decreasing permeability
- Increased by the thickness of the piping layer
- Increased by presence of an underlying layer of higher permeability
- Increased by increased horizontal to vertical permeability ratio
- Slightly decreased by angularity of the particles
- Not changed by confining stress
- Increased for turbulent flow (Annandale 2007)

Terzaghi et al. (1996) indicate that the mechanics of piping "defy theoretical approach," and the "results of theoretical investigations into the mechanical effects of the flow of seepage serve merely as a guide for judgment." The analytical methods described in this appendix merely provide a starting point to help develop a list of more likely and less likely factors during an elicitation of probability estimates.

Critical Gradient for Initiation of a Pipe

Backward erosion piping will initiate when a “heave” or zero effective stress condition occurs in soils subject to upward through-seepage. The heave equation or critical exit gradient from Terzaghi (1943) is given by:

$$i_{cr} = \frac{\gamma_b}{\gamma_w}$$ **Equation D-6-E-1**

where γ_b = buoyant unit weight of the soil; and γ_w = unit weight of water.

Several researchers (e.g., Kovács 1981, van Rhee and Bezuijen 1992) have evaluated seepage exiting sloping surfaces, where lower exit gradients are required for initiation of erosion. Each reduces the “classical” Terzaghi heave equation for vertical upward seepage with horizontal exit faces.

An example of portrayal of analytical results is shown in Figure D-6-E-1. In this example, the critical gradient for vertical (upward) seepage at the downstream toe was estimated. Based on the estimated vertical (upward) exit gradients from a seepage analysis, this figure can be used to help develop a list or more likely and less likely factors for initiation of backward erosion piping as a function of headwater level.

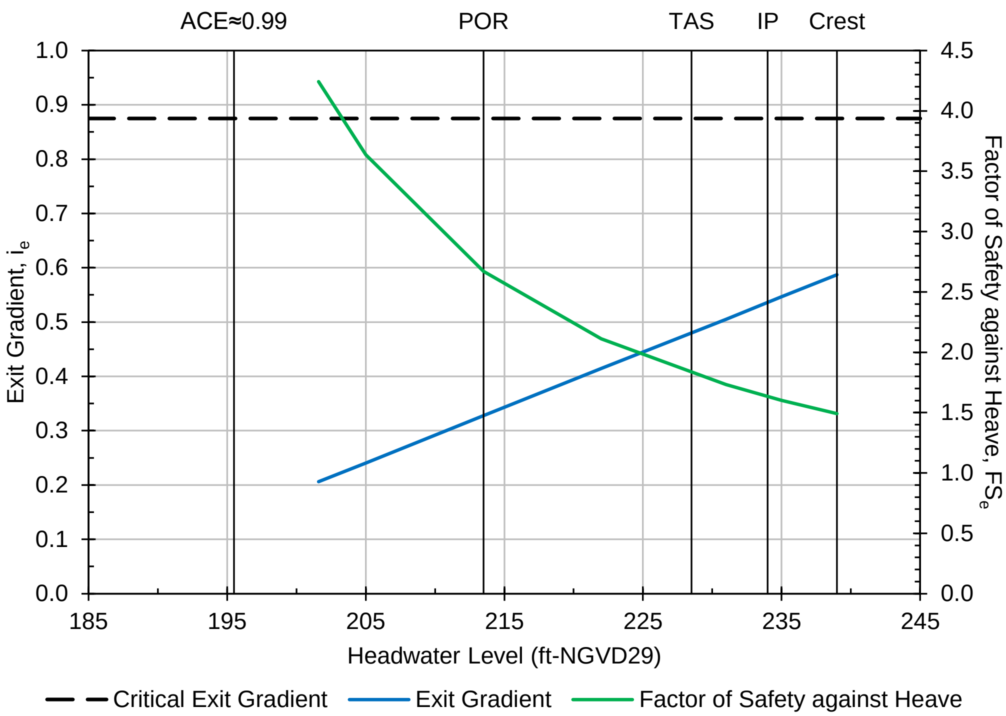


Figure D-6-E-1 Sample Portrayal of Analytical Results for Initiation of Backward Erosion Piping Taylor Series Method of Reliability Analysis

A Taylor series method of reliability analysis can be performed for selected random variables such as foundation layer thickness, permeability, unit weight, anisotropy, etc. The Taylor series provides probabilities of a factor of safety against heave of less than one for the reservoir level under consideration (i.e., probabilistic seepage analysis). For levees, USACE has performed such analyses in conjunction with blanket theory calculations since the 1990s.

Exceeding the limit-state condition simply provides an indication of the likelihood for backward erosion to initiate. Analytical results should be used to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

The methodology is described in ETL 1110-2-561 (31 January 2006). Estimates of the mean and standard deviations are required, and can be developed through an elicitation and application of the six-sigma rule, where the standard deviation is estimated:

$$\sigma = \frac{HCV - LCV}{6}$$ **Equation D-6-E-1**

where HCV = highest conceivable value; and LCV = lowest conceivable value. For the mean plus or minus three standard deviations, 99.73% of the area under the normal distribution is included. Therefore, essentially all of the values represented by the normal distribution curve are included.

An example of portrayal of Taylor series results for a given reservoir level is shown in Figure D-6-E-2. In this example, four random variables were considered: horizontal permeability of upper layer (Kha), horizontal permeability of lower layer (Khb), thickness of upper layer (Ta), thickness of lower layer (Tb), and P_I = probability of a factor of safety against heave less than one. For a given reservoir level, nine separate seepage analyses were performed for the combination of random variables shown to obtain an estimate of the exit gradient at the downstream toe of the embankment dam. The process was then repeated for the other reservoir levels.

Case	Random Variables			
	Kha (fpd)	Khb (fpd)	Ta (feet)	Tb (feet)
Mean, μ	40	500	10	80
Standard Deviation, σ = (HCV−LCV)/6	7.5	75	0	20

Critical gradient for particle detachment, i_{cr} 0.875

Run Case	Random Variables				i_e	FS_e	$Var(FS_e)$
	Kha (fpd)	Khb (fpd)	Ta (feet)	Tb (feet)			
1	40	500	10	80	0.705	1.242	
2	32.5	500	10	80	0.793	1.103	0.019
3	47.5	500	10	80	0.635	1.378	
4	40	425	10	80	0.625	1.399	0.018
5	40	575	10	80	0.775	1.129	
6	40	500	10	80	0.705	1.242	0.000
7	40	500	10	80	0.705	1.242	
8	40	500	10	60	0.601	1.455	0.028
9	40	500	10	100	0.781	1.121	

$\sigma_{FS} = [\Sigma Var(FS)]^{0.5}$ 0.255

$V_{FS} = \sigma_{FS} / E(FS)$ 0.205

$\beta = \ln[E(FS)/(1+V_{FS}^2)^{0.5}]/[\ln(1+V_{FS}^2)]^{0.5}$ 0.96

$P_I = P(FS < 1) = \Phi(-\beta)$ 1.68E-01

Figure D-6-E-2 Sample Taylor Series Results for Probabilities of a Factor of Safety against Heave Less than One

Critical Gradient for Progression of a Pipe

Bligh (1910) and Lane (1935)

Line-of-creep methods such as Bligh (1910) and Lane (1935) are still in use by some practitioners. They can be used for screening-level assessment of the critical gradient for progression of a pipe. Both empirical methods involve estimating the seepage path length beneath concrete structures (weirs) including cutoff walls. For application to embankment dams and levees, the seepage path length would be beneath the roof-forming material including upstream and downstream blankets or berms, cutoff walls, cutoff or inspection trenches, etc. The creep ratio is calculated as the total seepage path length divided by the hydraulic head difference. For Lane's method, the horizontal seepage path lengths are weighted 3 times less than the vertical seepage path lengths. Hence, it is often referred to as a "weighted creep" method.

$$C = \frac{L_1 + W + L_2 + 2D}{h} \text{ for Bligh} \quad \textbf{Equation D-6-E-2}$$

$$C_w = \frac{(L_1+W+L_2)/3+2D}{h} \text{ for Lane}$$ **Equation D-6-E-3**

where L_1 = length of upstream blanket or berm; W = width of base of embankment; L_2 = length of downstream blanket or berm; and d = depth of vertical structure (e.g., cutoff or weir).

To assess the likelihood of progression of backward erosion piping, the creep ratio for the reservoir level under consideration is compared to the minimum (or safe) creep ratio for the piping material in Table D-6-E-1. Progression of backward erosion would be expected if the creep ratio is less than the minimum creep ratio.

Table D-6-E-1 Minimum Creep Ratios

Piping Material	Bligh (1910)	Lane (1935)
Very fine sand or silt	18	8.5
Fine sand	15	7.0
Medium sand	#N/A	6.0
Coarse sand	12	5.0
Fine gravel	#N/A	4.0
Medium gravel	#N/A	3.5
Gravel and sand	9	#N/A
Coarse gravel, including cobbles	#N/A	3.0

The creep ratio is the reciprocal of the average gradient in the foundation for the reservoir level under consideration (i_{avf}), and the minimum creep ratio is the reciprocal of the critical gradient for progression of a pipe ($i_{adv} = 1/C$ or $i_{adv} = 1/C_w$).

Sellmeijer et al. (2011)

Sellmeijer et al. at Delft University of Technology (TU Delft) in The Netherlands developed a mathematical model for piping based on laboratory flume tests: Sellmeijer (1988), Sellmeijer and Koenders (1991), and Koenders and Sellmeijer (1992). The tests were performed mostly on fine to medium, uniform sands uniform ($1.58 \leq c_u \leq 3.53$) with some medium to coarse sands. Sellmeijer et al. (2011) extended and updated the piping model based on the results of several small-scale, seven medium-scale, and four large-scale field (IJkdijk) tests by Deltares / TU Delft reported in van Beek et al. (2009-10). The critical gradient for progression of a pipe is estimated as:

$$i_{adv} = F_R F_S F_G$$ **Equation D-6-E-4**

where F_R = resistance factor (strength of the layer subject to backward erosion); F_S = scale factor (relating pore size and seepage size); and F_G = geometrical shape factor.

The methodology applies to 2D seepage with plane or ditch exits parallel to the embankment for fine to medium sands within the limits of the test parameters shown in Table D-6-E-2. Van Beek et al. (2015) found that 3D configurations with flow towards a single point (e.g., hole in a confining layer) resulted in significantly smaller critical gradients than predicted by the model. In both small and medium-scale experiments, the model overestimated the critical gradient by a factor of approximately 2.

Table D-6-E-2 Parameter Limits during Piping Model Testing(Sellmeijer et al. 2011)

Parameter	Minimum	Maximum	Mean
Relative Density, RD (percent)	34	100	72.5
Coefficient of Uniformity, U	1.3	2.6	1.81
Roundness, KAS (percent)	35	70	49.8
Particle Size, d_{70} (mm)	0.150	0.430	0.208

Resistance Factor

The resistance factor (F_R) is calculated as:

$$F_R = \eta(G_s - 1)\tan(\theta)\left(\frac{RD}{72.5}\right)^{0.35}\left(\frac{U}{1.81}\right)^{0.13}\left(\frac{KAS}{49.8}\right)^{-0.02} \qquad \textbf{Equation D-6-E-5}$$

where KAS = roundness of the particles, which can be visually obtained using Figure D-6-E-3; RD = relative density (percent); U = coefficient of uniformity; G_s = specific gravity of soil particles; ϑ = bedding angle (deg); and □ = White's constant. The bedding angle and White's constant are held constant in Dutch practice with values of 37 degrees and 0.25, respectively.

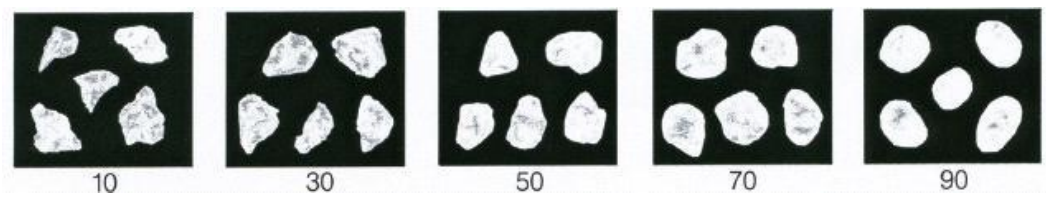


Figure D-6-E-3 KAS Indication of Angularity (van Beek et al. 2010)

Van Beek et al. (2010) indicate that KAS and U appear to be of less importance than the other sand characteristics, and have a weak influence on the critical gradient. Therefore, the U and KAS terms in the equation for F_R are sometimes ignored.

Scale Factor

The scale factor (F_S) is calculated as:

$$F_s = \frac{d_{70}}{(\kappa L)^{1/3}}\left(\frac{0.000208}{d_{70}}\right)^{0.6} \qquad \textbf{Equation D-6-E-6}$$

where d_{70} = particle size (m) for which 70 percent is finer (by weight); L = seepage path length (m) through the piping layer (measured horizontally); and κ = intrinsic permeability (m^2) of the piping layer which can be estimated by:

$$\kappa = k_h \frac{\mu}{\gamma_w}$$ **Equation D-6-E-7**

where μ = dynamic viscosity of water (N s/m^2); and k_h = permeability of the piping layer in the horizontal direction (m/sec).

Van Beek et al. (2012) adapted Sellmeijer's piping rule to multi-layer foundations to assess the influence of a coarse layer beneath the piping layer. The intrinsic permeability in the above equation is replaced with a layer-weighted average calculated as follows:

$$k_{h,avg} = \sum_{i=1}^{n} \frac{k_{h,i} D_i}{D}$$ **Equation D-6-E-8**

where D = total aquifer thickness.

<u>Geometrical Shape Factor</u>

The geometrical shape factor (F_G) is calculated as:

$$F_G = 0.91 \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1} + 0.04}$$ **Equation D-6-E-9**

where D = thickness of the piping layer (m); and L = seepage path length (m) through the piping layer (measured horizontally).

Schmertmann (2000)

Schmertmann (2000) carried out backward erosion piping tests in flumes at the University of Florida. The tests were carried out on a range of soils from fine to medium sands, up to coarse sand and fine gravel mixes. The soils were mostly fairly uniform($1.5 \le c_u \le 6.1$). He also plotted the Delft tests and found a similar correlation. Since the test geometries used at University of Florida and Delft were not the same, correction factors for geometry were applied in order to plot all of the results together. ***The methodology requires quite large corrections for scale effects***

and foundation geometry and applies to 2D seepage with plane or ditch exits parallel to the embankment centerline.

Laboratory Horizontal Critical Gradient

Schmertmann (2000) provided a linear relationship for estimating the horizontal critical gradient as a function of coefficient of uniformity based on study averages from flume tests. Robbins and Sharp (2016) examined the individual test results from each experimental series and provided the best-fit median (50th percentile) relationship shown in Figure D-6-E-4. Schmertmann's original relationships is labeled as the "no-test default line" in this figure and represents a lower bound than an average trend for low coefficients of uniformity ($c_u < 3$), which are of primary interest for backward erosion piping. For larger values of coefficient of uniformity, there is considerable more scatter in the data.

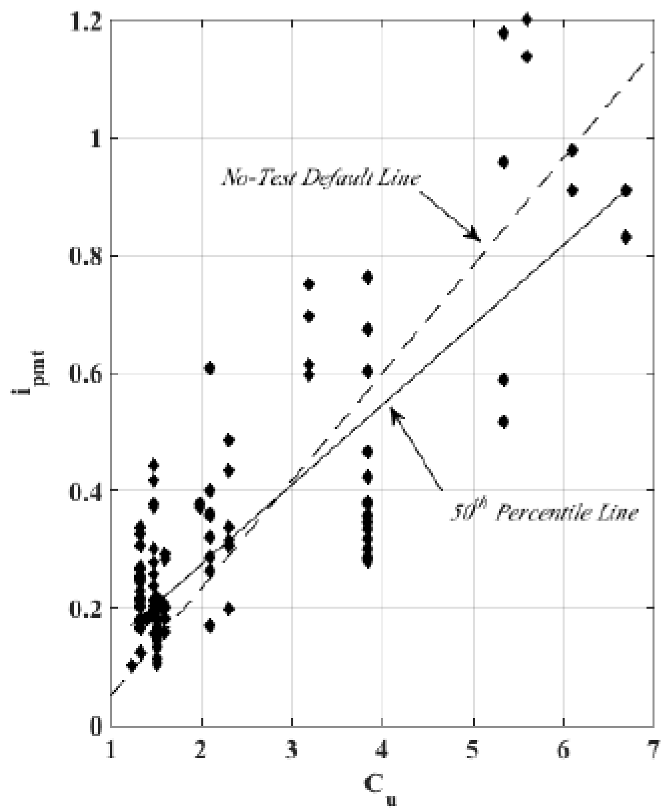


Figure D-6-E-4 Critical Point Gradients and Best-Fit Median Line (Robbins and Sharp 2016)

Several corrections for a number of factors are applied to the laboratory horizontal critical gradient to obtain the field horizontal critical gradient:

$$i_{po} = \left(\frac{C_D C_L C_S C_K C_\gamma C_Z}{C_R}\right) i_{pmt}$$

Equation D-6-E-10

where C_D = correction factor for depth/length ratio; C_L = correction factor for total pipe length; C_S = correction factor for grain-size; C_K = correction factor for anisotropic permeability of layer subject to backward erosion; C_{γ} = correction factor for density; C_Z = correction factor for high-permeability underlayer; C_R = correction factor for dam axis curvature; and i_{pmt} = maximum point seepage gradient needed for complete piping in the flume test based on the soil's coefficient of uniformity from Figure D-6-E-4. Additional information about the correction factors is provided below. ***Some errors contained in Schmertmann (2000) were corrected.***

The laboratory testing essentially used clean sands. No sands with silty fines and no sand-gravel mixtures were apparently used. These materials could behave differently than the limited range of sands that were used in the flume tests. In addition, controlled laboratory testing may not adequately account for actual field variability, and the large number of correction factors that are applied for field conditions suggest the tests may not adequately cover cases encountered in the field. ***Careful evaluation of the appropriateness of the method for a specific dam or levee is needed.***

<u>Depth/Length Ratio Factor</u>

The D/L_f factor (C_D) can be determined from Figure D-6-E-5, where D = thickness of the piping layer measured perpendicular to the flow lines (i.e., perpendicular to pipe inclination, α); and L_f = direct (not meandered) length between ends of a completed pipe path, from downstream to upstream exit, measured along the pipe path on a transformed section. For a vertical flow path, $D/L_f = \infty$ and $C_D = 0.715$. For horizontal flow paths, D = the vertical thickness of the piping layer. For steeply inclined flow paths, interpolate between these limits. Schmertmann (2000) amended the Weijers and Sellmeijer (1993) theory to obtain the relationship for the D/L_f factor shown in Figure D-6-E-5 and calculated as:

$$C_D = \frac{\left(\frac{D}{L_f}\right)^{\frac{0.2}{\left(\frac{D}{L_f}\right)^{2}-1}}}{1.4}$$

Equation D-6-E-11

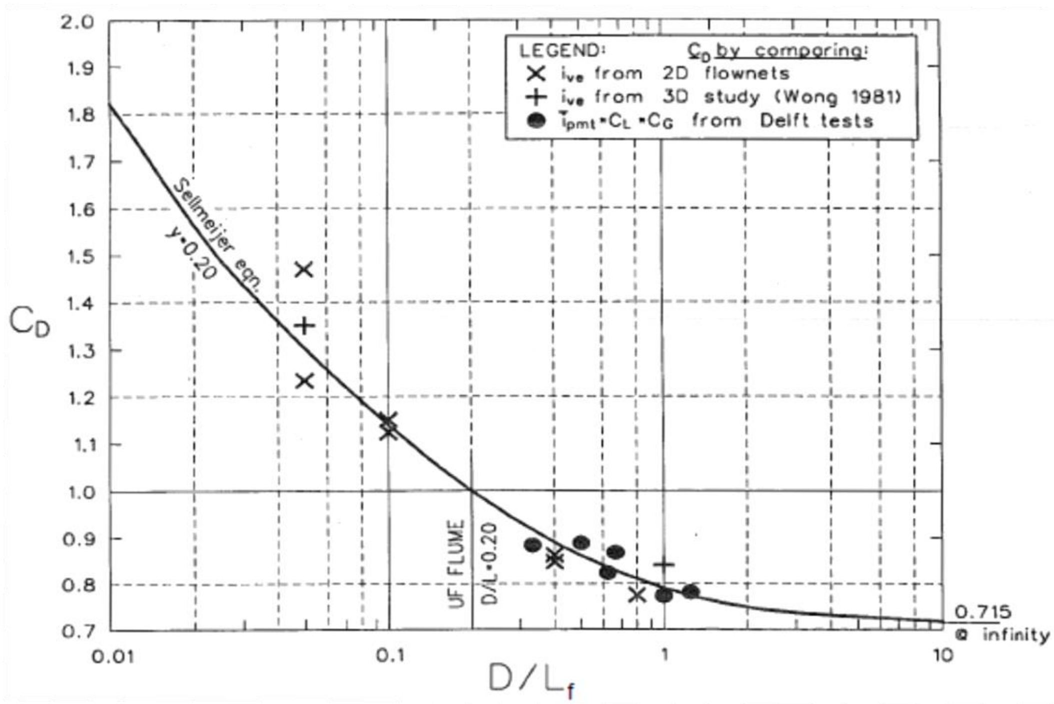


Figure D-6-E-5 Correction Factor for Depth/Length Ratio (Schmertmann 2000)

<u>Length Factor</u>

The length factor (C_L) is calculated as:

$$C_L = \left(\frac{L_t}{L_f}\right)^{0.2}$$

Equation D-6-E-12

where L_t = flume model length (L_t = 5 feet if Figure D-6-E-4 is being used to estimate i_{pmt}), where

$$L_f = \frac{L}{\left(\frac{k_h}{k_v}\right)^{0.5}}$$ **Equation D-6-E-13**

where k_h = permeability of the piping layer in the horizontal direction; and k_v = permeability of the piping layer in the vertical direction; and L = direct (not meandered) length (feet) between ends of a completed pipe path, from downstream to upstream exit, measured along the pipe path.

Grain-Size Factor

The grain-size factor (C_S) is calculated as:

$$C_S = \left(\frac{d_{10f}}{0.20 \text{ mm}}\right)^{0.2}$$ **Equation D-6-E-14**

where d_{10f} = particle size (mm) of the (field) piping layer for which 10 percent of the total weight is finer.

Anisotropic Permeability Factor

The anisotropic permeability factor (C_K) is calculated as:

$$C_K = \left(\frac{1.5}{R_{kf}}\right)^{0.5}$$ **Equation D-6-E-15**

where R_{kf} = anisotropy of the piping layer (k_h/k_v).

Density Factor

The density factor (C_γ) is calculated as:

$$C_{\gamma} = 1 + 0.4\left(\frac{D_{rf}}{100} - 0.6\right)$$ **Equation D-6-E-16**

where D_{rf} = relative density of soil layer subject to backward erosion.

Underlayer Factor

If the layer susceptible to piping is underlain by a high-permeability underlayer, Figure D-6-E-6 is used to determine the underlayer factor (C_Z), where D = thickness of the underlayer (feet); k_p = permeability of the piping layer; k_u = permeability of the underlayer; and r = equivalent radius (feet) of the developing pipe cross section (prior to gross enlargement). Schmertmann used small radii in his tests (0.3 inch and 0.6 inch). For practical purposes, r is very small, and D/r is very large, so it is suggested that C_Z = 1. If very thin erodible layers are being considered, use radii of 2.5 to 10 cm. For thin alternating layers of erodible and non-erodible soil modeled as a homogenous layer with high anisotropy, use C_Z = 1.

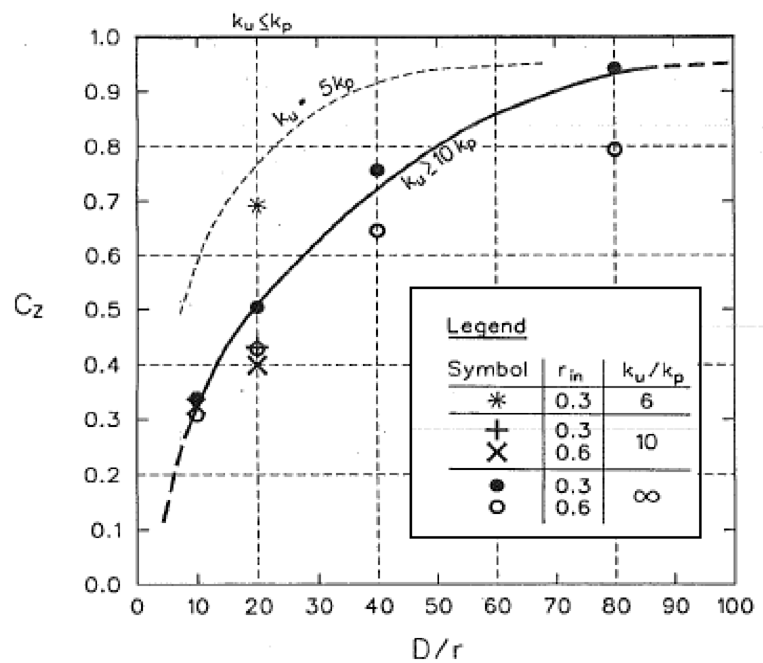


Figure D-6-E-6 Correction Factor for High-Permeability Underlayer (Schmertmann 2000)

Gradient Factor for Convergent/Divergent Flow

The correction factor for dam axis curvature (C_R) is calculated as:

$$C_R = 1.0 \text{ for a straight dam alignment} \quad \textbf{Equation D-6-E-17}$$

Or

$$C_R = \frac{R_1 + R_0}{2R} \text{ for a curved dam axis} \quad \textbf{Equation D-6-E-18}$$

where R = radius to point on the pipe path in a dam with curved axis (i.e., radius of curvature in the dam); R_0 = shortest radius to an end of completed pipe path (i.e., distance from the center of curvature to the upstream toe; and R_1 = longest radius to an end of completed pipe path (i.e., distance from the center of curvature to the downstream toe).

Pipe Inclination Adjustment

The field horizontal critical gradient is then adjusted for pipe inclination using the pipe inclination adjustment (C_α) which is calculated as:

$$C_\alpha = \frac{i_{p\alpha}}{i_{po}} \quad \textbf{Equation D-6-E-19}$$

where $i_{p\alpha}$ = field critical gradient for the angle (α) of the advancing pipe path (towards the impounded water) from Figure D-6-E-7. If the pipe path progresses upward, α is positive, whereas α is negative if the pipe path progresses downward. For a horizontal seepage exit α = 0 degrees, and for a vertical seepage exit α = -90 degrees. Figure D-6-E-8 can be used as a guide for determination of the sign for α.

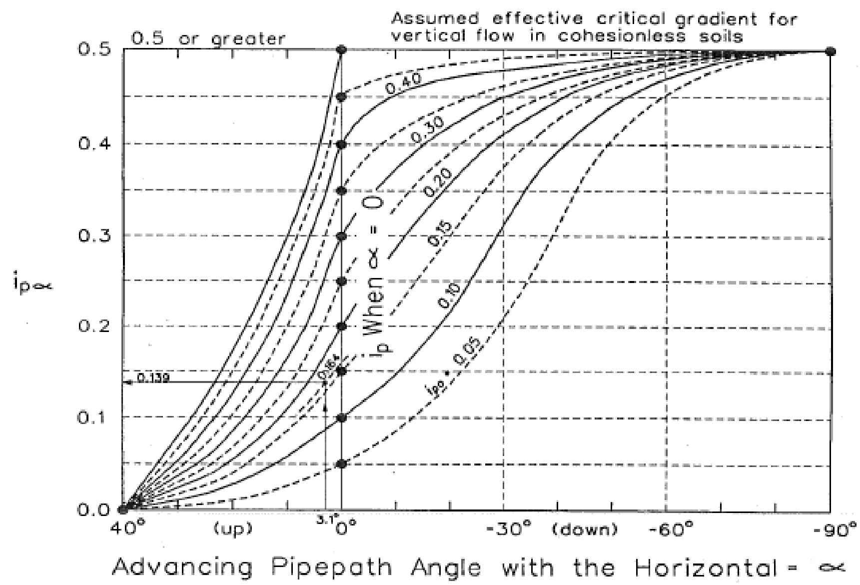


Figure D-6-E-7 Field Critical Gradient (Schmertmann 2000)

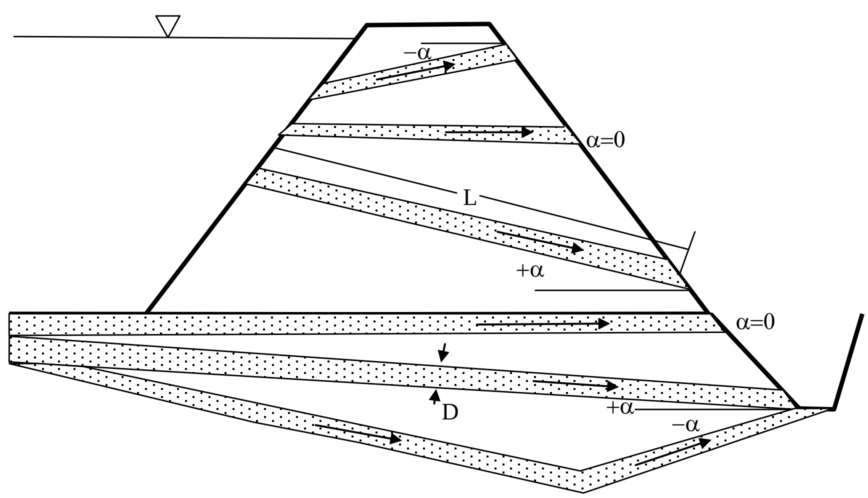


Figure D-6-E-8 Pipe Path Inclination Geometry

<u>Field Critical Gradient for Progression of the Pipe</u>

The field critical gradient for progression of the pipe is then obtained by applying the pipe inclination adjustment to the field horizontal critical gradient:

$$i_{adv} = i_{po} C_{\alpha}$$ **Equation D-6-E-20**

Hydraulic Condition for Progression of a Pipe

To assess the likelihood of progression of a pipe (hydraulic condition), the average gradient for the reservoir level under consideration is compared to the critical gradient for progression of a pipe. An example of portrayal of analytical results for multiple methods is shown in Figure D-6-E-9. In this example, the critical gradient for progression of a pipe was evaluated using four methods. Based on the estimated average gradient (hydraulic head difference divided by the seepage path length), this figure can be used to help develop a list of more likely and less likely factors for the hydraulic condition for progression of backward erosion piping as a function of reservoir level. The methods shown may not be given equal weight by the risk team in assessing the probability.

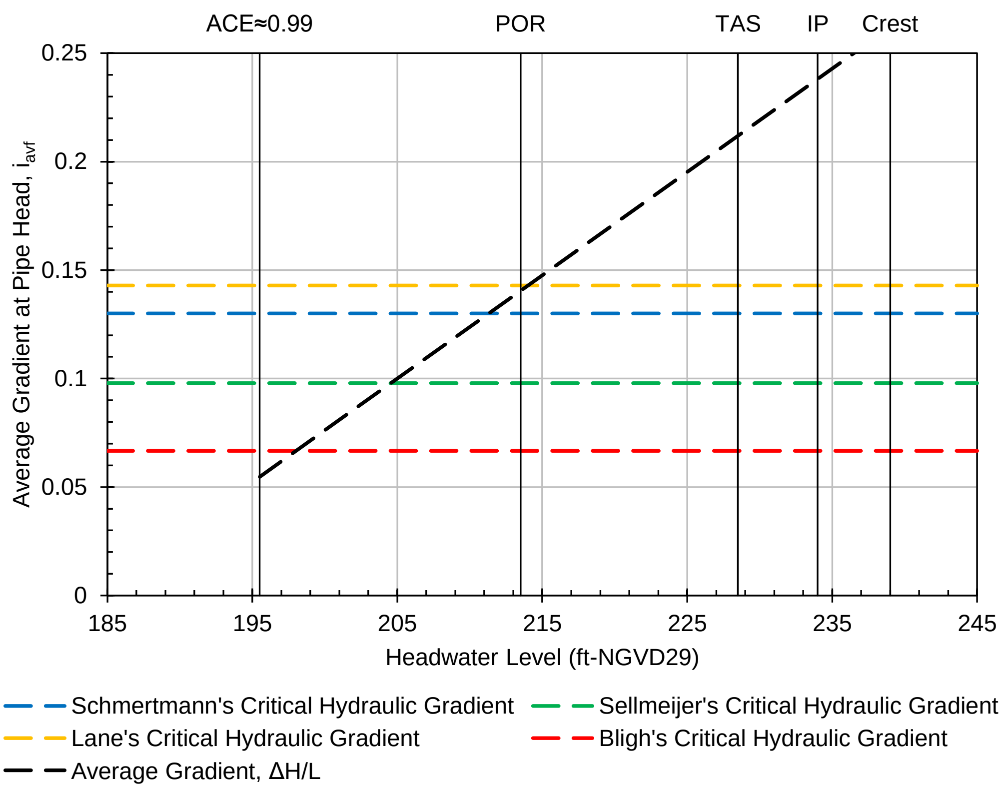


Figure D-6-E-9 Sample Portrayal of Analytical Results for

Likelihood of Progression of a Pipe (Hydraulic Condition)
Sensitivity or uncertainty analysis is recommended. In addition to a best estimate, a range of values should be considered from a reasonable low estimate to a reasonable high estimate. Probability distributions can also be assigned for the various input parameters to be used in a Monte Carlo simulation to assess the probability of a factor of safety against progression of the pipe (hydraulic condition).

Robbins and Sharp (2016) presented the results of a best-fit quantile regression analysis from the individual laboratory flume tests. Figure D-6-E-10 can be used to develop a cumulative density function to estimate the probability of backward erosion piping progressing. An example of portrayal of analytical results obtained using the Robbins and Sharp best-fit quantile regression lines is shown in Figure D-6-E-11.

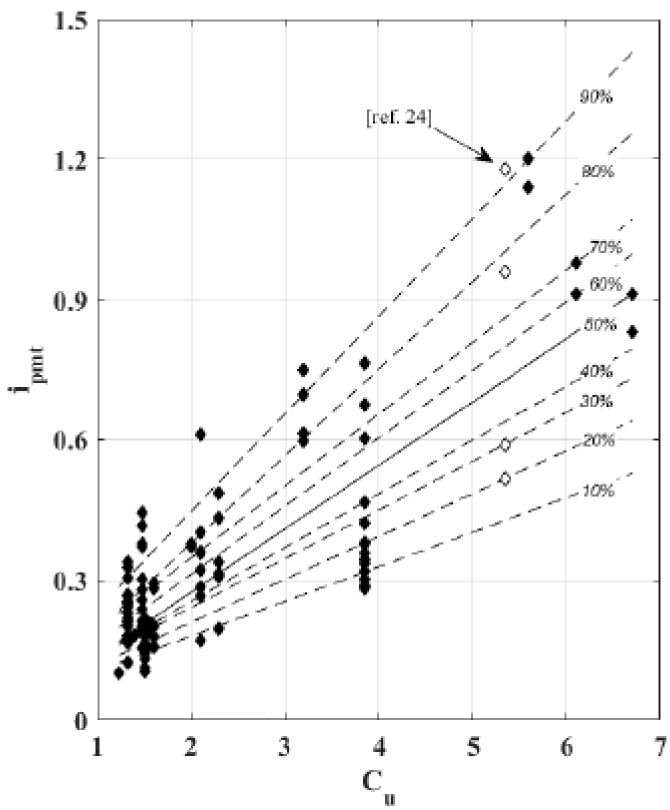


Figure D-6-E-10 Critical Point Gradients and Best-Fit Quantile Regression Lines (Robbins and Sharp 2016)

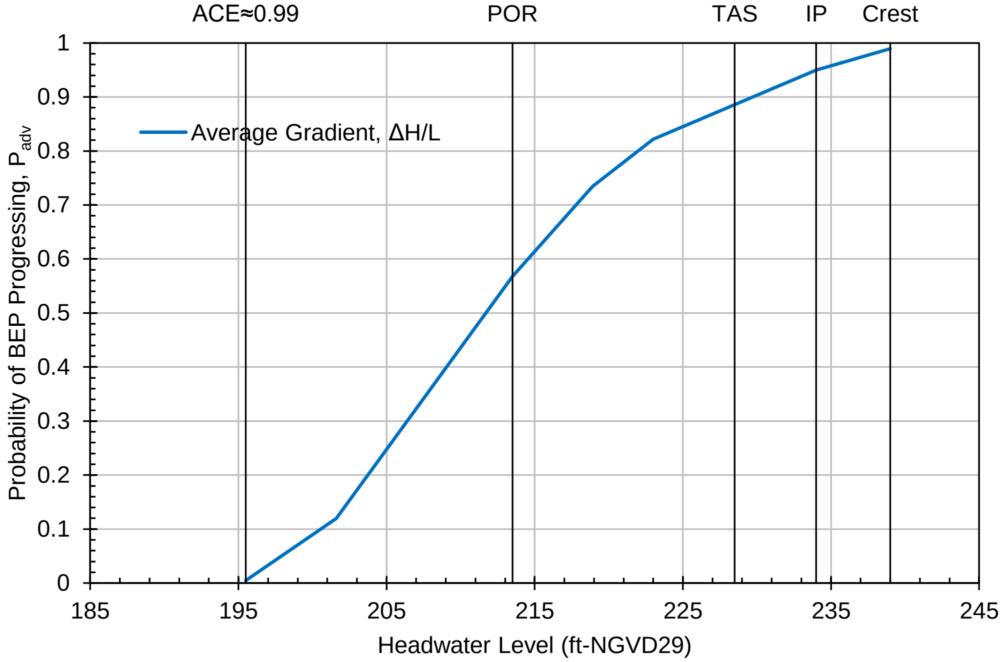


Figure D-6-E-11 Sample Portrayal of Analytical Results for Likelihood of Progression of a Pipe (Hydraulic Condition)

Analytical results should be used to help to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

Appendix D-6-F: Internal Instability (Suffusion)

Geometric Condition (Detailed Evaluation of Susceptibility)

If the screening-level review of the gradation curves indicates the soil is potentially internally unstable, then the more robust methods in in this appendix may be applied to further evaluate the susceptibility to internal instability.

Several methods are described, and some methods may not apply to the materials under consideration. Multiple methods are suggested to help inform judgment. Marot et al. (2014) made the following suggestions for assessing the geometric criteria:

- Use Kézdi's criterion for gap-graded soils.
- Use Kenney and Lau's criterion for broadly graded soils with F < 15%.
- Use Kézdi's criterion for F > 15%, per Li and Fannin (2008).
- Use Wan and Fell's criteria (alternative method) for broadly graded silt-sand-gravel soils with F > 15%, per Marot et al. (2014).

Hydraulic Condition

There is little published literature on the seepage gradient required to initiate suffusion. Skempton and Brogan (1994) investigated the hydraulic criterion for the erosion of fine particles in well-graded and gap-graded sandy gravels and observed critical hydraulic gradients far less than the theoretical critical gradient for "heave."

Fell et al. (2004) summarized some general observations from laboratory testing:

- Soils with a higher porosity start to erode at lower hydraulic gradients.
- Soils with clayey fines erode at relatively higher hydraulic gradients than soils without clayey fines at similar fines contents.
- Soils with higher soil density erode at higher critical gradients, given the fines content of the soils are the same.
- Gap-graded soils erode at a relatively lower critical gradients than non-gap-graded soils with similar fines content.

According to Marot et al. (2014), the hydraulic loading on the particles is often described by three different approaches:

- Hydraulic gradient: Skempton and Brogan (1994) and Li (2008)
- Hydraulic shear stress: Reddi et al. (2000)
- Pore velocity: Marot et al. (2011, 2012)

However, more research is needed with a wider range of soils, hydraulic gradients, and flow orientation. In many of the internally stable soils tested in the laboratory, the gradients required to initiate suffusion were so high that they are unlikely to occur in dams, levees, or their foundations.

Burenkova (1993)

Based on the results of laboratory testing on cohesionless sand-gravel soils with maximum particle sizes up to 100 mm and coefficients of uniformity up to 200, Burenkova (1993) proposed a geometric condition for internal stability of a soil that depends on the conditional factors of uniformity ($h' = d_{90}/d_{60}$ and $h'' = d_{90}/d_{15}$) as shown in Figure D-6-F-1.

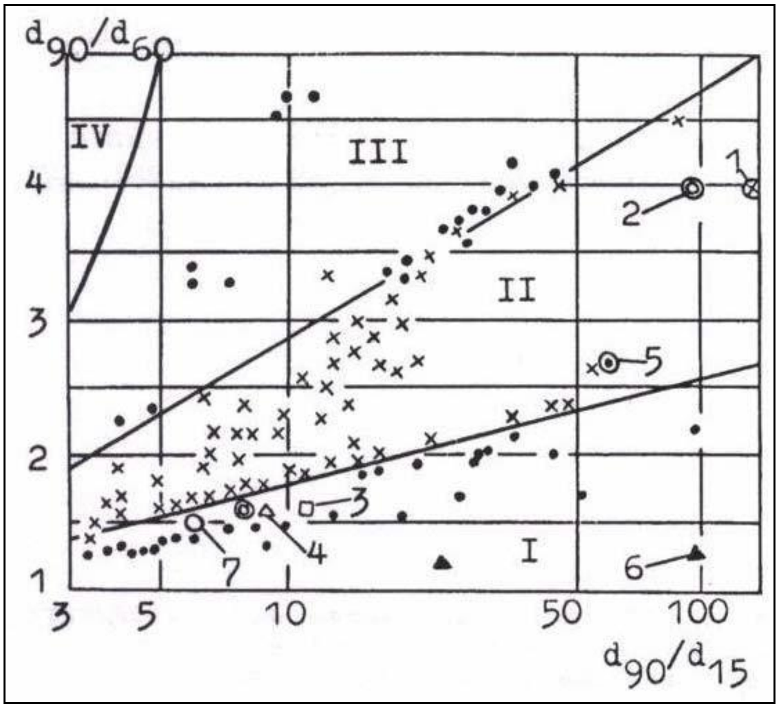


Figure D-6-F-1 Materials Susceptible to Internal Instability (Burenkova 1993)

Boundaries were defined separating “suffusive soils” from “non-suffusive soils”. Zones I and III represent zones of suffusive compositions; Zone II represents a zone of non-suffusive compositions; and Zone IV represents a zone of artificial soils. Zone II (non-suffusive) boundaries are defined as follows:

$$0.76\log(h'') + 1 < h' < 1.86\log(h'') + 1 \qquad \textbf{Equation D-6-F-1}$$

Wan and Fell (2004a, 2008)

According to Wan and Fell (2004a, 2008), the Burenkova (1993) method did not give a clear boundary between internally stable and unstable soils in the data set. Therefore, they developed contours for predicting the probability of internal instability by logistic regression of h′ and h″. Their “modified Burenkova method” for broadly graded and gap-graded soils is shown in Figure D-6-F-2 for silt-sand-gravel and clay-silt-sand-gravel mixtures of limited plasticity and clay content (i.e., PI ≤ 12 and less than 10 percent clay-size fraction, defined as the percentage finer than 0.002 mm) and Figure D-6-F-3 for sand-gravel mixtures with a non-plastic FC < 10 percent. The contours in Figure D-6-F-3 predict higher probabilities of internal instability than those in

Figure D-6-F-2 because the more erosion resistant clayey and silty soil samples were excluded from the data set. The probability contours are represented by the following equations (Wan and Fell 2004a):

$$P_I = \frac{e^Z}{1 + e^Z}$$ **Equation D-6-F-2**

For silt-sand-gravel soils and clay-silt-sand-gravel soils percent of limited clay content and plasticity,

$$Z = 2.378 \log(h'') - 3.648h' + 3.701$$ **Equation D-6-F-3**

For sand-gravel soils with less than 10 percent non-plastic fines,

$$Z = 3.875 \log(h'') - 3.591h' + 2.436$$ **Equation D-6-F-4**

The probabilities should not be used directly in a risk assessment, but rather used to help develop a list of more likely and less likely factors during an elicitation of probability estimates.

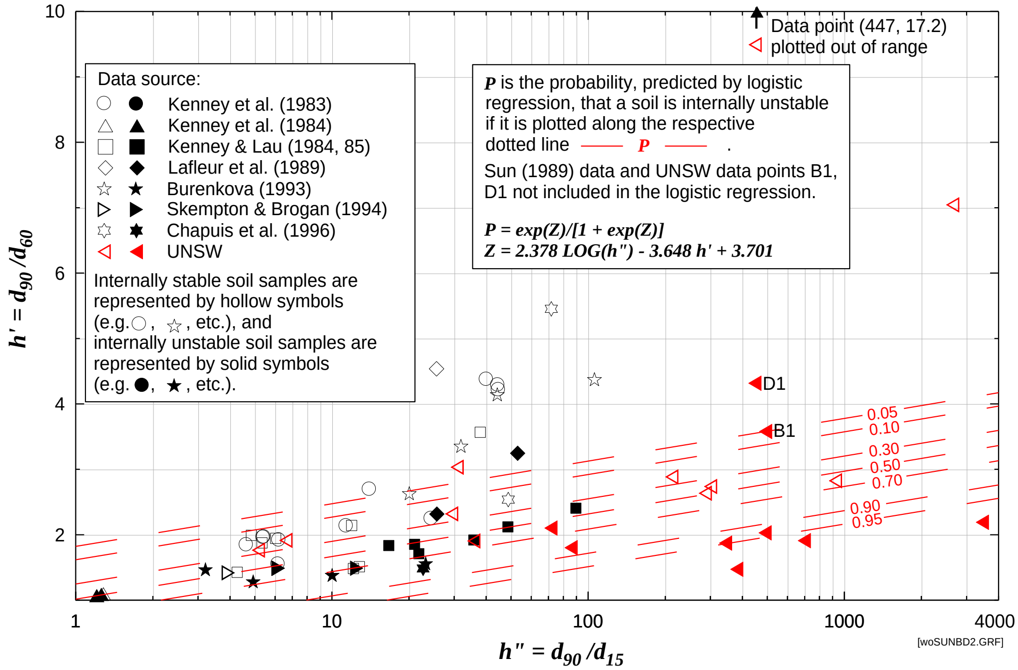


Figure D-6-F-2 Probability of internal instability for silt-sand-gravel soils and clay-silt-sand-gravel soils of limited clay content and plasticity (Wan and Fell 2004a)

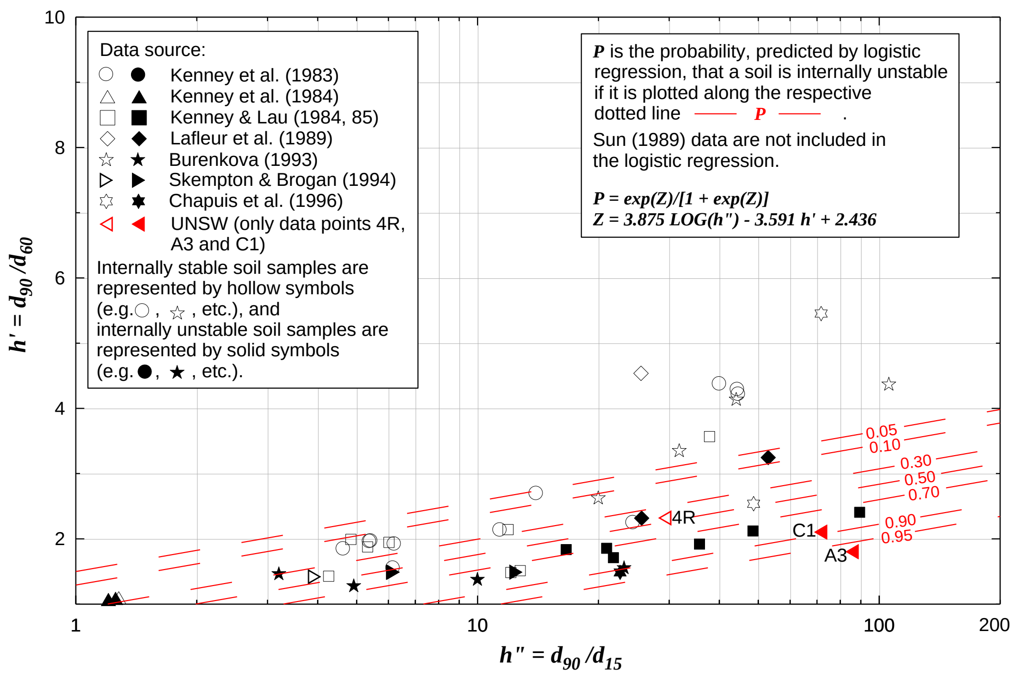


Figure D-6-F-3 Probability of internal instability for sand-gravel soils (Wan and Fell 2004a)

Wan and Fell (2008)

Wan and Fell (2008) also proposed an alternative method for broadly graded silt-sand-gravel soils as a function of d_{90}/d_{60} and d_{20}/d_5. Boundaries shown in Figure D-6-F-4 were proposed for likelihood of internal instability. This method is not applicable to gap-graded soils.

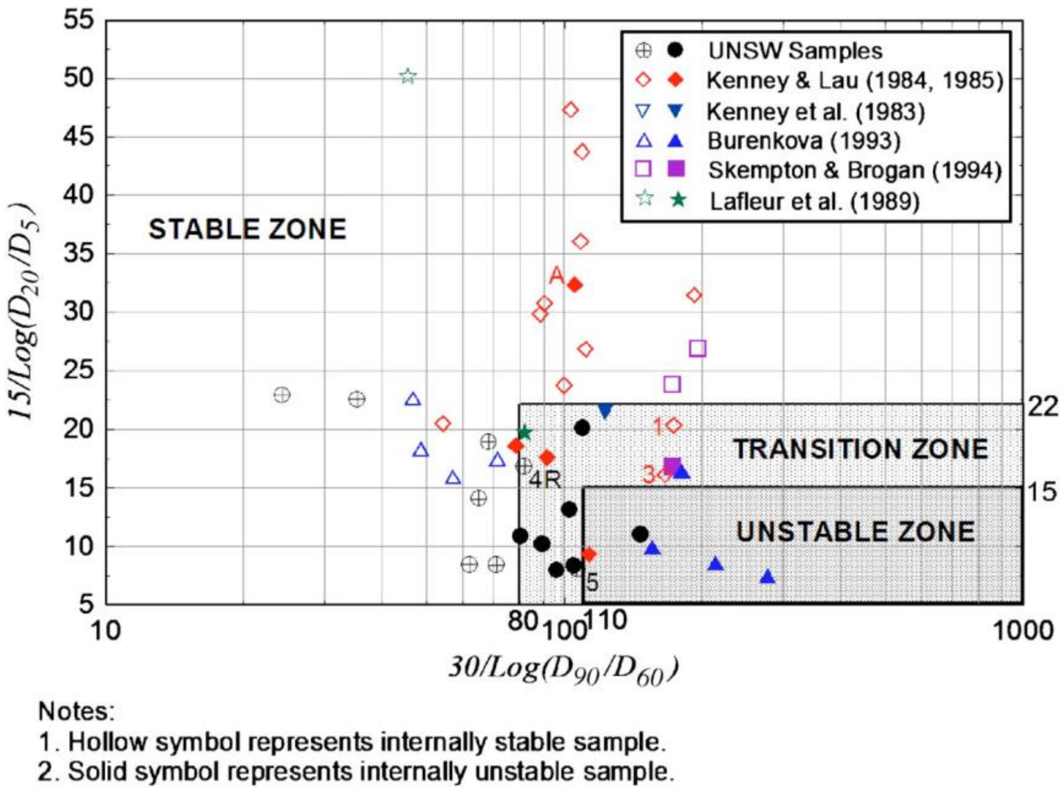


Figure D-6-F-4 Alternative Method for Assessing Internal Instability (Wan and Fell 2008)

Li and Fannin (2008)

Li and Fannin (2008) reviewed two commonly used methods to determine the susceptibility to internal instability: Kézdi (1979) and Kenney and Lau (1985, 1986). Kézdi divided a soil into a coarse fraction and a fine fraction at one point along its particle-size distribution curve and applied Terzaghi's (1939) rule for designing protective filters (D'_{15}/d'_{85}) to the two fractions, with the fine fraction as the "base" and the coarse fraction as the "filter," to assess if the soil would self-filter and be internally stable. The mass increment (H) over D'_{15} and d'_{85} is constant and equal to 15 percent, resulting in a criterion for instability of H less than 15 percent.

Kenney and Lau calculated an H/F stability index over the increment D to 4D, which increases in magnitude with progression along the gradation curve, where H is the mass fraction between D and 4D and F is the mass passing. They originally proposed a criterion in 1985 for internal instability of $H/F < 1.3$, applicable within $F \leq 30$ percent (and $c_u \leq 3$) for narrowly graded soils and within $F \leq 20$ percent (and $c_u > 3$) for widely graded soils. This criterion was subsequently revised in 1986 to $H/F < 1.0$. This method is commonly used for cohesionless sand-gravel soils (e.g., Reclamation's "4x" line).

An example of converting a particle-size distribution curve to H-F space (referred to as the "shape curve") is shown in Figure D-6-F-5:

$4D = 4 \times D$

$H = F_{4D} - F$

Gradation Curve

D (mm)	F (%)	4D (mm)	F_{4D} (%)	H (%)	H/F
182.288	99.0	729.152	99.0	0.0	#N/A
108.342	93.3	433.368	99.0	5.7	0.061
61.327	77.2	245.308	99.0	21.8	0.282
40.184	51.3	160.736	97.4	46.0	0.897
16.432	32.1	65.728	78.7	46.7	1.456
5.265	20.1	21.060	35.8	15.8	0.785
2.748	17.2	10.992	26.2	9.0	0.523
1.053	16.0	4.212	18.9	2.9	0.183
0.596	14.7	2.384	16.9	2.3	0.154
0.424	12.2	1.696	16.4	4.3	0.351
0.282	8.4	1.128	16.0	7.6	0.912
0.191	5.5	0.764	15.2	9.6	1.740
0.129	3.6	0.516	13.5	9.9	2.717
0.075	2.4	0.300	8.9	6.5	2.731
0.055	1.1	0.220	6.4	5.3	4.804

$4(40.184) = 160.736$

$16 - 8.4 = 7.6$

Figure D-6-F-5 Example of Obtaining the Shape Curve

Li and Fannin (2008) combined aspects of these two methods for assessing the susceptibility to internal instability. They concluded that the Kenney and Lau criterion is more conservative at $F > 15$ percent, but the Kézdi criterion is more conservative at $F < 15$ percent. The combined criteria are shown in Figure D-6-F-6, where the respective values of H and F are plotted at $(H/F)_{min}$.

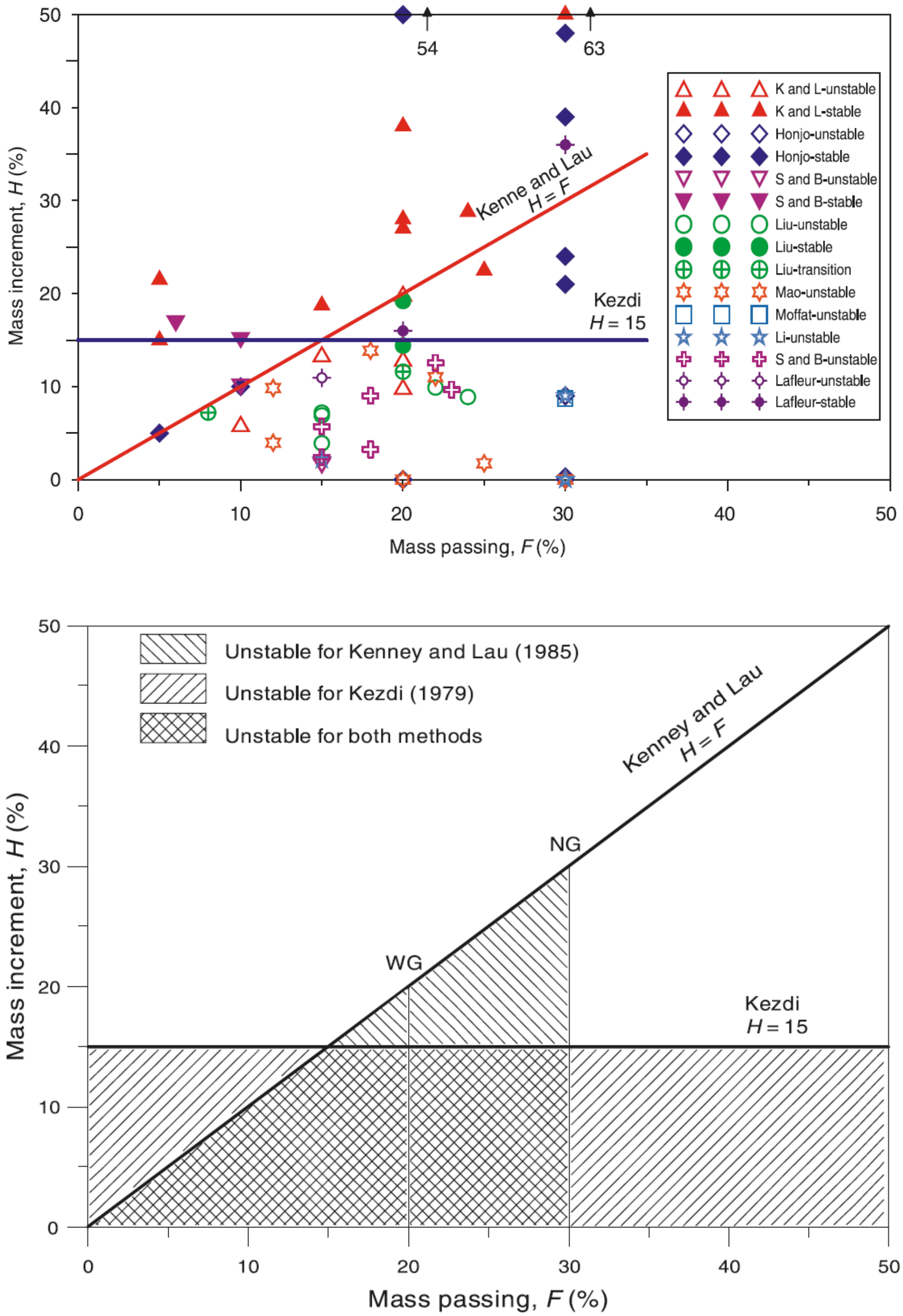


Figure D-6-F-6 Criteria for Internal Instability (Li and Fannin 2008)

Appendix D-6-G Detailed List of Conditions which Increase the Likelihood of Initiation of an Internal Erosion Process

In consideration of Fell et al (2008), the following is a detailed list of conditions that increase the likelihood of initiation of the various processes of internal erosion. Combinations of some of these conditions are common in case histories of incidents involving internal erosion and should be considered where appropriate.

Leads to increased likelihood of scour:

Through Embankment

Cracking of embankment from differential settlement of fill

- Wide benches or “stair steps” in the upper to middle portion of the abutment profile
- Steep abutments near the top of the embankment
- Very steep abutments and a narrow valley can lead to “arching” of the soil across the valley leading to a reduction in vertical confining stress within the embankment and increased potential for hydraulic fracturing (i.e., pore pressures exceed confining stress).
- Fell et al. (2008) suggest that differential settlement between the shell and the core (if deformability of the materials differ) can lead to “dragging and transverse shearing” of the core. However, more typically, this type of differential settlement leads to longitudinal cracks at the interface between the two materials. The core, if more deformable than the shells, can get “hung up” leading to transverse near horizontal low stress zones and cracking through the core.
- Seasonal shut-downs or placement in freezing weather can lead to a pervious layer through the core if not properly treated (i.e., frozen material and desiccation cracking was not removed and the surface thoroughly scarified with good moisture control upon re-compaction). In the unlikely event that post-shutdown construction results in lower modulus material in comparison to the underlying embankment, differential settlement of the overlying embankment can lead to transverse cracking.
- Desiccation of the embankment material in upper part of the core or at a shutdown.
- Excessive settlements as a percentage of the embankment height (i.e., more than about 3 to 5 percent during construction or about 1 percent at 10 years post-construction) increases the chances of transverse cracking – even lesser settlements may lead to cracking in particularly

brittle soils. Note that cracking is often masked; case histories suggest that such cracking can go unnoticed for years and even decades.

- Poor core density due to lack of formal compaction, lack of compaction control, or excessively thick compacted layers can result in cracks or defective layers through the core.

Cracking of embankment due to diff. settlement seated in foundation

- Different foundation conditions (deformability) across the profile.

- A narrow steep-walled cutoff trench forms a location where arching of core material placed into the trench can lead to a low density zone in the core susceptible to transverse hydraulic fracturing.

- Low-density fine-grained loess soils or weakly cemented “desert” soils present within the foundation may collapse upon wetting. These conditions could lead to coincident cracks in foundation soils.

- Different deformability conditions between fill and foundation soils (such as at diversion channels)

- Excessive settlements as a percentage of the embankment height (i.e., more than about 3 to 5 percent during construction or about 1 percent at 10 years post-construction) increases the chances of transverse cracking – even lesser settlements may lead to cracking in particularly brittle soils. Note that cracking is often masked; case histories suggest that such cracking can go unnoticed for years and even decades.

- An irregular foundation contact surface, possibly with overhanging rock features, or sloppy or loose foundation soil conditions upon embankment placement

- Irregular rock surfaces and overhangs beneath foundation soils that are not cutoff can cause differential settlement and/or defects beneath overhangs.

Through foundation – soil filled joints. Silt and fine sand deposits against openwork gravels.

From the embankment into the foundation - Open joints, seams, faults, shears, bedding planes, solution features, or other discontinuities at contact with rock foundation. Silt embankment constructed directly upon openwork gravel.

- Ridges and valleys formed by excavation along geologic features (e.g., tilted bedding planes forming an irregular surface) that trend upstream to downstream, into which compaction is difficult, can lead to cracks/flaws pathways near the embankment-rock contact and hydraulic fracture (Quail Creek Dike).

- Open joints, seams, faults, shears, bedding planes, solution features, or other discontinuities in the rock foundation at the contact with the embankment core into which core material can erode, especially if the following also apply:

  - The discontinuities trend upstream to downstream across the foundation, providing a pathway for reservoir seepage.

  - There was no or questionable foundation surface treatment performed during construction in the way of dental concrete or slush grout, especially if the treatment area was narrow with respect to the height of the embankment.

  - The effectiveness of foundation grouting is questionable due to grout holes being parallel to open discontinuities, poor grout mixes, widely-spaced holes with uncertain closure, uncaulked surface leaks during grouting, and/or little pore-pressure drop across the grout curtain as measured by piezometers.

  - The discontinuities are open, or perhaps filled with erodible silty or sandy material. Wider discontinuities are more problematic than narrow ones.

- Poor clean-up at the core-foundation rock surface can lead to a flaw.

- Highly permeable gravel foundation materials which can transmit significant flow capable of eroding material at the base of the embankment and carrying it downstream.

Associated with structures - A conduit through the embankment can creates a potential crack due to the potential for inadequate density or compaction, especially if one or more of the following conditions are also present:

  - A round conduit with no concrete encasement resulting in poor compaction

  - The presence of seepage cutoff collars

  - Cracks or open joints in the conduit, or corrugated metal pipe which is subject to corrosion deterioration and through-going holes,

  - Steep and narrow trench into which the conduit was placed, which makes compaction difficult and creates the potential for arching of soil across the trench, leaving a low density zone susceptible to hydraulic fracturing.

- A stiff conduit projecting up into a brittle embankment

- An outlet conduit trench forms a location where arching of core material placed into the trench can lead to a low density zone in the core susceptible to transverse hydraulic fracturing.

- Presence of frost-susceptible soils in which ice lenses can form, particularly when these materials are adjacent to conduits or other structures that could increase the possibility of freezing conditions.

- Against a spillway wall, due to difficulties in compacting against the wall (especially if vertical or counterforted), Settlement away from the wall parallel to the abutment, can potentially lead to a high permeability zone or small gap adjacent to the wall.

- For composite concrete/embankment dams, vertical faces, overhangs, and changes in slopes of the concrete section (against which the embankment core is compacted) can lead to cracks/flaws especially if post-construction embankment settlements are large.

- Direct observations such as observed transverse cracks or settlement of fill adjacent to structures

- Concentrated seepage or wet areas on the downstream face of the embankment, adjacent to a structure be indications that flaw may extend through the embankment.

Leads to increased likelihood of internal migration

Through or in Embankment – broadly graded core placed in contact with coarse grained fill. Typically happens along a steep contact.

- Evidence of sinkholes or depressions (especially along the alignment of a penetrating outlet works conduit), could be indications that material has moved by means of seepage flows.

Through the foundation – fine grained soils over coarse grained open work soils – such as reservoir sediment over glacial outwash gravels.

From the embankment into the foundation - Untreated open joints, seams, etc in foundation rock overlain by non-cohesive embankment fill.

- Open joints, seams, faults, shears, bedding planes, solution features, or other discontinuities in the rock foundation at the contact with the embankment into which fill can drop into or be eroded down into.

Along or into Structures – damaged conduit overlain by non-cohesive fill.

Into Drains – broken drain pipes or inadequately designed drain systems overlain by non-cohesive embankment fill.

Leads to increased likelihood of backwards erosion piping

Through Embankment – severe filter incompatibility at core contact with shell.

- Rodent holes and root balls, if not properly treated, can be locations for piping to initiate. Rodents may burrow into dry areas of an embankment when the reservoir is low, but these areas may be exposed to the reservoir as it rises. Similarly, decaying root systems can form pathways for piping initiation.

Through the foundation – continuous fine sand deposits, natural impervious blanket overlying fine sands hydraulically connected to river or reservoir.

- A low permeability confining layer at the toe of the embankment beneath which high artesian pressures exist, which increases the chance of blowout.
- Sand boils observed in the channel downstream of the embankment which could be indications of material movement associated with a foundation seepage path, especially if material is moving out away from the boils.

From the embankment into the foundation - core material placed directly against gravels in bottom of downstream side of cutoff trench

- Embankment core material placed against the downstream slope of a cutoff trench cut into pervious gravels with no intervening filter leaves an interface through which core material can be eroded.

Along or into Structures – damaged conduit surrounded with non-cohesive embankment or founded upon fine sands or silts

Into Embankment Drains or Structural drains – damaged or unfiltered drain pipe at downstream end of continuous sand or silt deposits.

Leads to increased likelihood of internal instability

Through Embankment – non-plastic broadly graded core against coarse downstream shell

Through the foundation – gap-graded soil deposits such as some glacial deposits

Appendix D-6-H: Continuation

Susceptibility to Cracking

The ability of a filter material to hold a crack generally depends on the fines content, cementation, or the presence of plastic fines. Filters with low fines content and non-plastic fines are generally less likely to sustain a crack than filters with a high fines content comprised of plastic fines. Criteria have been developed to decrease the amount of fines and thus the chance of a filter cracking and it is important to include these when evaluating an existing filter. For example, Reclamation and USACE filter design criteria require a minimum D_5F equal to 0.075 mm (i.e., non-plastic fines content less than or equal to 5 percent) in the final in-place product to help ensure these filters will not hold a crack. To achieve the maximum allowable fines content after compaction, the "off the belt" at the quarry/crusher stockpile typically had about 3 percent fines to account for breakdown during handling, transportation, placing, and compacting. In some circumstances for critical modern designs, the maximum in-place fines content has been limited to 3 percent. Reclamation and USACE filter design criteria also require the portion of the filter material passing the No. 40 (0.425 mm) sieve be non-plastic (i.e., PI = 0). Cementation increases the likelihood of cracking. Typical cementing agents include carbonate materials (e.g., limestone or dolomite), gypsum, sulfide materials, and volcanic (pyroclastic) ash, particularly for sand-sized particles. Even small amounts of silt in broadly graded, silty sandy gravel transition zones or filters may result in cracking. As suggested by Terzaghi and Peck, a dense well-graded transition zone with a slight amount of silt fines can crack. There is some laboratory evidence that thin (less than 5 feet thick), vertical, clean, partially saturated and compacted filters subject to severe cracking may hold a crack, and a gravel zone downstream of a cracked filter allowed for healing of the cracked filter (Redlinger et al. 2011). Table D-6-H-1, which is based on laboratory testing conducted by Park (2003) and field performance data from Foster (1999) and Foster and Fell (1999), provides guidance on assessing the likelihood of a filter material holding a crack. ***The descriptors should be used to help develop a list of more likely and less likely factors during a team elicitation of probability estimates.***

Table D-6-H-1 Likelihood of a Material Holding a Crack (adapted from Fell et al. 2004)

Plasticity of Fines	Fines Content, FC (percent)	Likelihood of Holding a Crack	
		Well Compacted	Not Compacted
Non-plastic (and no cementing present)	5 to 7	Unlikely	Very Unlikely
	7 to 15	Likely	Unlikely to Likely
	≥ 15	Very Likely	Likely
Plastic (or fines susceptible to cementing)	5 to 7	Likely	Unlikely to Likely
	7 to 15	Very Likely	Likely
	≥ 15	Virtually Certain	Very Likely

The evaluation for cracking of a filter or transition zone also needs to consider the effect of stress conditions and the presence of flaws or defects in this zone along with consideration of "common causes" for a flaw in the impervious zone. Consideration of "common causes" using the bullet lists of conditions that may lead to an increased likelihood of a flaw existing <u>through the dam</u> (including considerations for conduits through the dam), a flaw <u>through the foundation</u> or <u>from the embankment into the foundation</u> contained earlier in this chapter should be included.

Susceptibility to Segregation

Segregation is the tendency of large particles in a given mass of aggregate to gather together whenever the material is being stockpiled, loaded, transported, placed or otherwise disturbed. Segregation of filter material can cause pockets of coarse zones that may not be filter-compatible with the material being protected. For segregation to be a significant contributor to the likelihood of continuation of internal erosion, an entire lift of the filter zone has to be segregated from upstream to downstream, which is very unlikely except for very narrow zones, and the segregated layer has to correspond with a flaw or concentrated seep in the embankment. For narrow filter zones placed upstream to downstream in one pass, it may be necessary to evaluate the potential for segregation. A common cause of segregation is improper material handling. Material placed in a pile off of a conveyor, or loaded from a chute, or from a hopper segregates because the larger particles roll to the sides of the stockpiles or piles within the hauling unit.

Material dumped from a truck, front loader, or other placing equipment almost always segregates, with the severity of the segregation corresponding to the height of the drop, moisture content, and the maximum size of the particles. Soils which are susceptible to internal instability are also susceptible to segregation during placement which aggravates the problem as coarse particles become nested in a matrix of finer particles.

Based on laboratory testing, Kenney and Westland (1993) concluded that all dry soils consisting of sands and gravels segregate in the same general way, independent of grain size and grain size distribution. Dry soils containing particle sizes smaller than 0.075 mm segregate to a smaller extent than soils not containing fines, and water in sandy soils (mean size finer than 3 mm to 4 mm) inhibits segregation but has little influence on the segregation of gravels (mean size coarser than 10 to 12 mm). To minimize segregation during construction, Reclamation and USACE filter design criteria, which limits the amount of fines and oversize material, as shown in Table D-6-H-2, can be used to help evaluate existing filter/transition zones. Although a minimum D_5F size of 0.075 mm may have been specified in the final in-place product, breakdown may occur during placement and compaction. The filter design criteria also limits the maximum allowable $D_{90}F$ size based on the minimum $D_{10}F$ size, as shown in Table D-6-H-3.

Table D-6-H-2 Minimum and Maximum Particle Size Criteria for Filters (adapted from FEMA 2011)

Base Soil Category	Minimum D_5F	Maximum $D_{100}F$
All Categories	≥ 0.075 mm (No. 200 sieve)	≤ 2 inches (75 mm)
Note: USACE (2005) sets maximum $D_{100}F$ at 3 inches (75 mm), maximum FC of 5 percent, and PI of zero.		

Table D-6-H-3 Segregation Criteria for Filters (adapted FEMA 2011)

Base Soil Category	If Minimum $D_{10}F$ is: (mm)	Then Maximum $D_{90}F$ is: (mm)
All Categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

Estimated Gradation after Segregation or Washout

Fell et al. (2008) recommended an approximate method for estimating the $D_{15}F$ of filter materials after segregation or washout that assumes 50 percent of the finer soil fraction is segregated out or 50 percent of the unstable or erodible soil fraction is washed out. However, Fell (2016) indicated that it is more appropriate to assume 100 percent.

Evaluation of Filters (or Adjacent Materials) not Meeting Modern Particle Retention Criteria

Filter zones and adjacent materials which are coarser than required by modern design methods based on particle size will often be quite effective in controlling internal erosion (Foster and Fell 1999, 2001). Downstream rockfill and sand/gravel zones which were not designed as filters may provide some protection against continuation of internal erosion. In addition, foundation soils can also provide some protection against continuation. Depending on the ratio of particle and pore sizes, the erosion will either:

- Not continue (i.e., no erosion); or
- Stop after only minor erosion (i.e., some erosion); or
- Stop only after a significant amount of erosion (i.e., excessive erosion); or
- Continue (i.e., continuing erosion)

These erosion filter erosion boundaries are conceptually shown in Figure D-6-H-1.

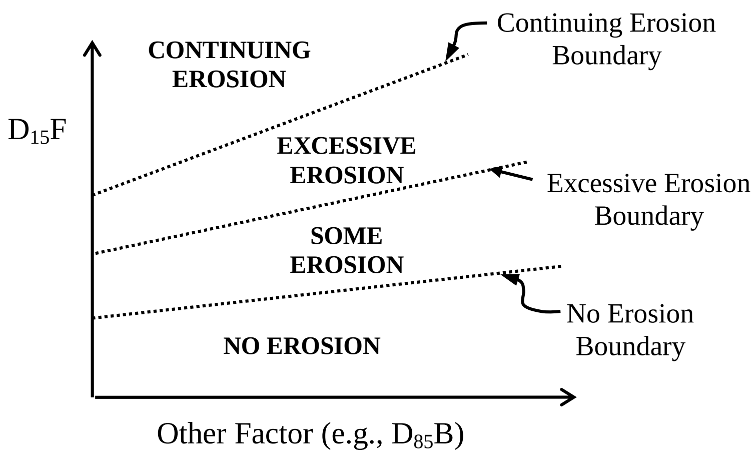


Figure D-6-H-1 Conceptual Filter Erosion Boundaries (Foster 1999 and Foster and Fell 2001)

The filter evaluation relies heavily on the work of Foster and Fell (2001) to determine no erosion, some erosion, excessive erosion, and continuing erosion boundaries for the base soil. Continuing erosion indicates the base soil could be eroded through the filter without plugging off, and this is the primary focus of the evaluation of the likelihood of continuation of internal erosion. Although internal erosion is expected to initiate for some erosion and excessive erosion, it would eventually plug-off, given time under conditions in the laboratory. The filter testing performed was setup for a vertical downward flow regime. Different orientations in the field need to be considered with caution, especially into the sides of conduits.

Dividing the event tree into branches leading to breach for each of the erosion categories can be considered, particularly if the excessive-continuing erosion portion is high. In addition, if the likely breach mechanism cannot be judged with confidence during the PFMA, estimating the breach probability later could be difficult if the understanding of the mechanism for each erosion category is widely different.

Although Fell et al. (2008) suggest that each of the erosion categories be carried through the event tree, Reclamation and USACE practice has been to come up with one estimate of the likelihood of an unfiltered exit (as discussed earlier), for which a filter evaluation is just one aspect. It is typical to assign the probability of an unfiltered exit based on not just the likelihood of the continuing erosion (CE) boundary, but also *considering* the likelihood of the excessive erosion (EE) boundary, as well as *considering* how far the material is from no erosion (NE) boundary. Reclamation and USACE also consider the variability of the gradations (from fine to coarse extremes), how thick the filtering unit is, how continuous it is likely to be and whether it extends to a free or open face.

Although fairly prescriptive, the assessment is similar to traditional filter evaluation but with more steps, and it can provide a better indication of the likelihood of the core material being filtered even when modern "no erosion" filter criteria are not met in all cases. An example is provided at the end of this chapter. If sufficient gradation exists, the filter evaluation involves the steps described below. If gradation data does not exist or is limited, gradations can be estimated based on the likely source of the materials and any processing, as described in Fell et al. (2008).

- Select representative gradations of the original (or re-graded) base soil (i.e., coarse, average, and fine base soil gradations) based on the fine and coarse base soil envelopes from all gradation tests. For example, if the representative base soil gradation corresponds to 80 percent of all gradation tests, then the fine base soil gradation is indicative of the coarser 10 percent of the base soils, and the fine base soil gradation is indicative of the finer 10 percent of the base soils.

- Assess the no erosion (NE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations using Table D-6-H-4. For highly dispersive soils (pinhole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower $D_{15}F$ for the no erosion boundary, as shown in Table D-6-H-5 based on modern particle retention criteria.

Table D-6-H-4 Criteria for No Erosion Boundary for Non-Dispersive Soils (adapted from FEMA 2011)

Base Soil Category	Fines Content (percent)	Criteria for No Erosion Boundary
1	$FC > 85$	$D_{15}F \leq 9(D_{85}B)$
2	$40 < FC \leq 85$	$D_{15}F \leq 0.7$ mm
3	$15 < FC \leq 40$	$D_{15}F \leq (4(D_{85}B) - 0.7)\left(\frac{40 - FC}{25}\right) + 0.7$ If $4(D_{85}B) < 0.7$ mm, use $D_{15}F \leq 0.7$ mm.
4	$FC \leq 15$	$D_{15}F \leq 4(D_{85}B)$
Notes: The fines content is the percentage finer by weight than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.		

Table D-6-H-5 Criteria for No Erosion Boundary for Dispersive Soils (adapted from FEMA 2011)

Base Soil Category	Fines Content (percent)	Criteria for No Erosion Boundary
1	$FC > 85$	$D_{15}F \leq 6.5(D_{85}B)$
2	$35 < FC \leq 85$	$D_{15}F \leq 0.5$ mm
3	$15 < FC \leq 35$	$D_{15}F \leq (4(D_{85}B) - 0.5)\left(\frac{40 - FC}{25}\right) + 0.5$ If $4(D_{85}B) < 0.5$ mm, use $D_{15}F \leq 0.5$ mm
4	$FC \leq 15$	$D_{15}F \leq 4(D_{85}B)$
Notes: The fines content is the percentage finer by weight than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.		

Assess the excessive erosion (EE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations using Table D-6-H-6.

Table D-6-H-6 Criteria for Excessive Erosion Boundary (adapted from Foster and Fell 1999, 2001)

Base Soil	Criteria for Excessive Erosion Boundary
$D_{95}B \leq 0.3$ mm	$D_{15}F > 9(D_{95}B)$
$0.3 < D_{95}B \leq 2$ mm	$D_{15}F > 9(D_{90}B)$
$D_{95}B > 2$ mm and FC ≤ 15 percent	$D_{15}F > 9(D_{85}B)$
$D_{95}B > 2$ mm and 15 percent < FC ≤ 35 percent	$D_{15}F > 2.5\left((4(D_{85}B) - 0.7)\left(\frac{35 - FC}{20}\right) + 0.7\right)$
$D_{95}B > 2$ mm and FC > 35 percent	$D_{15}F >$ ($D_{15}F$ value for erosion loss of $0.25 g/cm^2$ in the CEF test, as shown in Figure D-6-H-2, can be estimated as $D_{15}F \approx 0.34(1.07)^{fm}$ by curve-fit)

Notes: Criteria are directly applicable to soils with $D_{95}B$ up to 4.75 mm. For soils with coarser particles, determine $D_{85}B$ and $D_{95}B$ using gradation curves adjusted to give a maximum size of 4.75 mm.

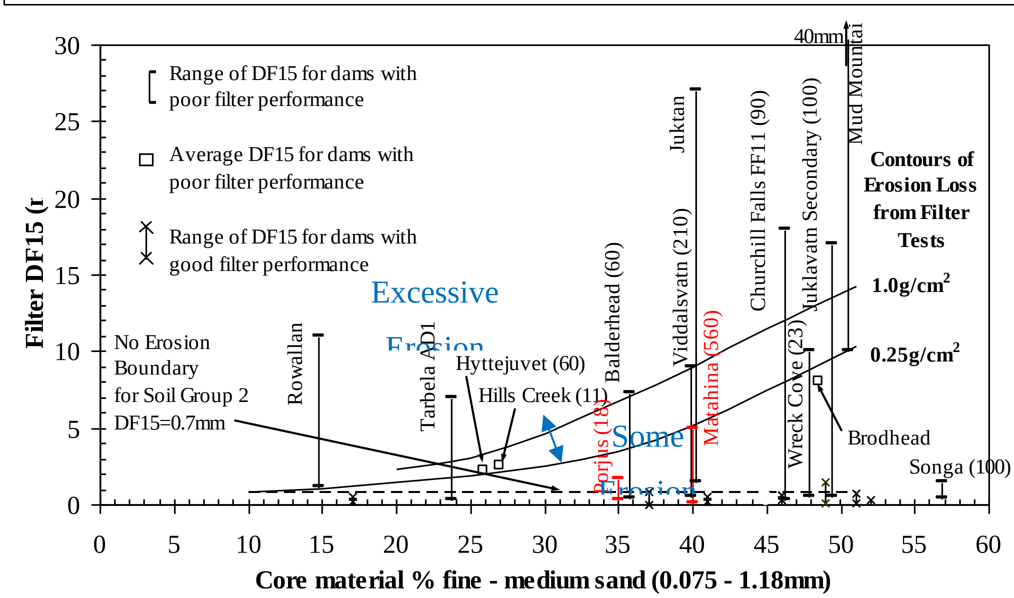


Figure D-6-H-2 Criteria for Excessive Erosion Boundary (adapted from Fell et al. 2008)

Assess the continuing erosion (CE) boundary based on the actual (or re-graded) base soil for the coarse, average, and fine base soil gradations. For all soils, this is estimated as $D_{15}F > 9(D_{95}B)$ (Foster and Fell 1999, 2001).

- Plot the erosion boundaries on the original filter gradation curves (and the adjusted filter gradation curves for segregation or washout) on the D_{15} line.

- Estimate the proportion of the original filter gradation (and filter gradation after segregation or washout) within each of the erosion categories for the coarse, average, and fine base soil gradations. The suggested approach is to estimate the proportions for the continuing, excessive, and some erosion categories first and then calculate the proportion for the no erosion category by subtracting the sum of the other proportions from one.

Coarse base soil gradation: $P_{NE,\text{ coarse}} = 1 – (P_{CE,\text{ coarse}} + P_{EE,\text{ coarse}} + P_{SE,\text{ coarse}})$

Average base soil gradation: $P_{NE,\text{ average}} = 1 – (P_{CE,\text{ average}} + P_{EE,\text{ average}} + P_{SE,\text{ average}})$

Fine base soil gradation: $P_{NE,\text{ fine}} = 1 – (P_{CE,\text{ fine}} + P_{EE,\text{ fine}} + P_{SE,\text{ fine}})$

- Make an initial estimate of the probabilities of no erosion, some erosion, excessive erosion, and continuing erosion by calculating the sum-product of the percentage of base soil gradations and the estimated percentage of no erosion, some erosion, excessive erosion, and continuing erosion for the coarse, average, and fine base soil gradations. The calculations are as follows, where N corresponds to the representative base soil gradation (i.e., as a percentage of all gradation tests) and n = (100 –N)/2 corresponds to the percentage finer or coarser of the base soil:

$P_{NE} = (n/100)(P_{NE,\text{ coarse}}) + (N/100)(P_{NE,\text{ average}}) + (n/100)(P_{NE,\text{ fine}})$

$P_{SE} = (n/100)(P_{SE,\text{ coarse}}) + (N/100)(P_{SE,\text{ average}}) + (n/100)(P_{SE,\text{ fine}})$

$P_{EE} = (n/100)(P_{EE,\text{ coarse}}) + (N/100)(P_{EE,\text{ average}}) + (n/100)(P_{EE,\text{ fine}})$

$P_{CE} = (n/100)(P_{CE,\text{ coarse}}) + (N/100)(P_{CE,\text{ average}}) + (n/100)(P_{CE,\text{ fine}})$

- If the filter gradation is finer than the continuing erosion boundary, Fell et al. (2008) suggest using Table D-6-H-7 to estimate the probabilities of continuing erosion (based on how much finer the gradations are compared to the continuing erosion boundary) to allow for the possibility of the gradations being coarser than indicated by the available information. ***The probabilities should not be used directly in a risk assessment, but rather used to help develop a list likely of more likely and less likely factors during an elicitation of probability estimates.***

Table D-6-H-7 Probability of Continuing Erosion when the Actual Filter Gradation Is Finer than the Continuing Erosion Boundary (adapted from Foster and Fell et al. 2008)

$D_{15}F$	P_{CE}
$< 0.1(D_{15}F_{CE})$	0.0001
$< 0.2(D_{15}F_{CE})$	0.001
$< 0.5(D_{15}F_{CE})$	0.01 – 0.05

Example Filter Evaluation

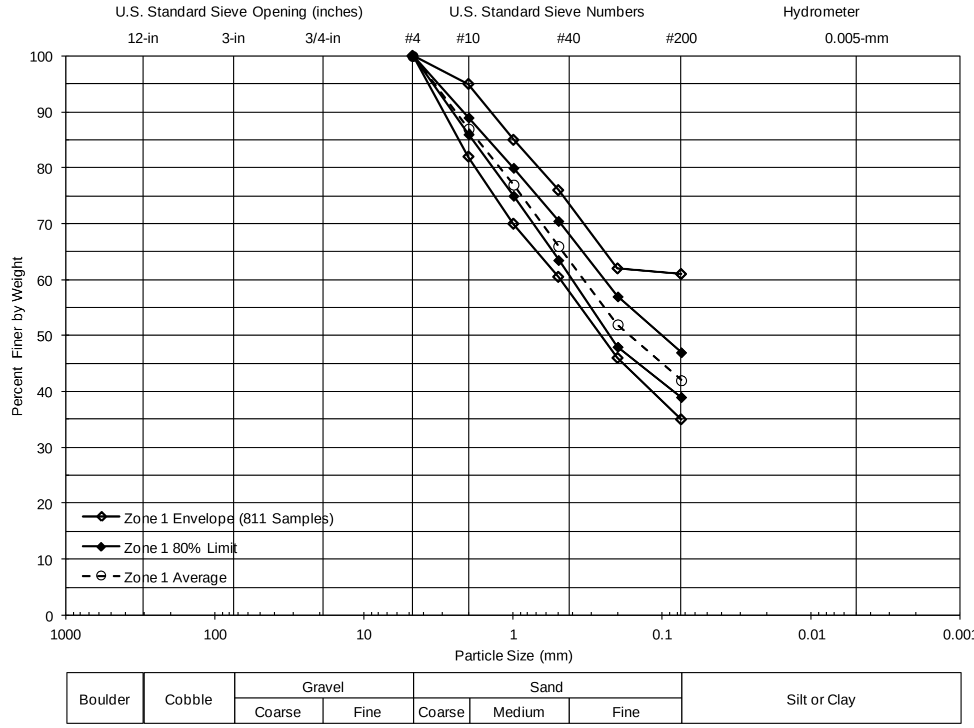


Figure D-6-H-3 Example Re-Graded Base Soil

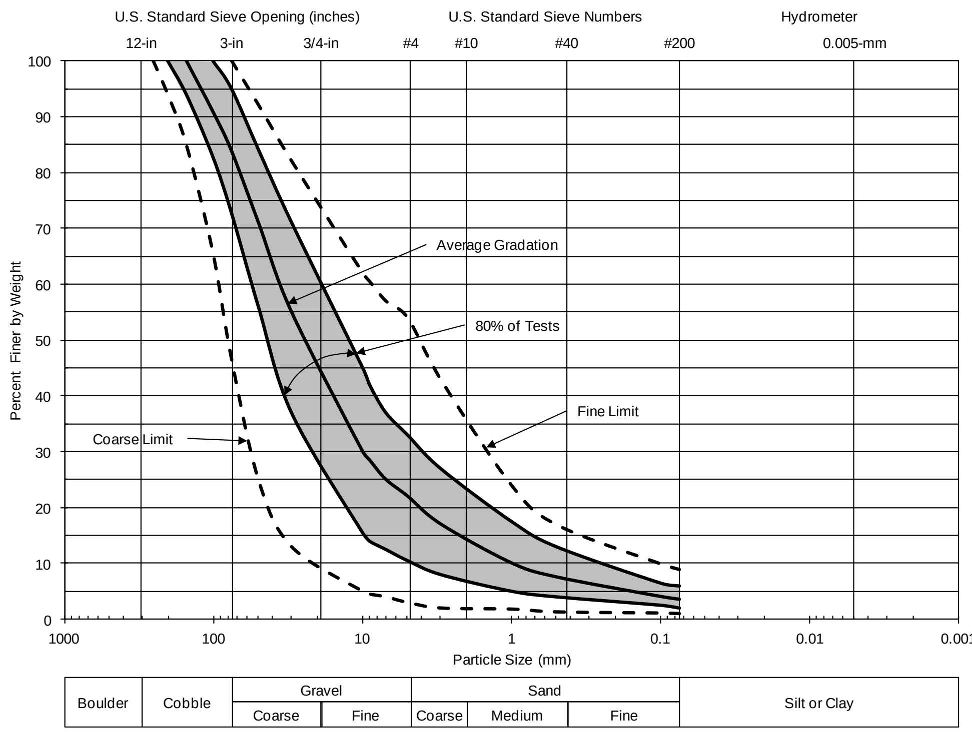


Figure D-6-H-4 Example Filter Gradations

- Assess if the filter materials are susceptible to cracking. The fines content of the representative boundary filter gradations in Figure D-6-H-4 is between about 2 and 6 percent. Based on Table D-6-H-1, the likelihood of the filter material holding a crack would be small, especially for non-plastic fines.

- Assess if the filter materials are susceptible to segregation. Based on Table D-6-H-3, the boundary filter gradations in Figure D-6-H-4 indicate the limits to prevent segregation are not met by a large margin. The minimum $D_{10}F$ of about 0.25 mm correspond to a maximum $D_{90}F$ to prevent segregation of 20 mm. However, the maximum $D_{90}F$ is actually about 120 mm. Using the average filter gradation, the minimum $D_{10}F$ is about 1 mm, and the maximum $D_{90}F$ is about 95 mm. The maximum allowable $D_{90}F$ is more like 30 mm, and again the criteria to limit segregation are not met.

- Assess if the filter materials are susceptible to internal instability. Based on Reclamation criteria, the ratio of say the $D_{80}F$ to the $D_{10}F$ particle sizes is much greater than 4, and the likelihood of internal instability appears to be small. However, the filter gradation curve was judged to have a flat tail of fines, which may be susceptible to internal instability.

- Estimate the gradation after segregation or washout. The adjusted gradation curve is shown on Figure D-6-H-6. This example assumes 50 percent washout of the finer fraction, and is possibly un-conservative.

- Assess the no erosion (NE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations. The re-graded fines content of the base soil in Figure D-6-H-3 is between 35 and 61 percent. Based on Table D-6-H-4, the fines content corresponds to Base Soil Category 2 and a no erosion boundary of $D_{15}F < 0.7$ mm.

- Assess the excessive erosion (EE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations. Based on Table D-6-H-6, the base soil is best classified as a soil with $D_{95}B > 2$ mm and FC > 35 percent. This requires determining the excessive erosion boundary from Figure D-6-H-2 using the percentage of material between 0.075 and 1.18 mm (defined as fine to medium sand). The results are summarized in Table D-6-H-8.

- Assess the continuing erosion (CE) boundary based on the actual (or re-graded) base soil for the coarse, average, and fine base soil gradations as $D_{15}F < 9(D_{85}B)$. The results are summarized in Table D-6-H-8.

Table D-6-H-8 Erosion Boundaries for Example Base Soil

Core Gradation	Base Soil Characteristics			No Erosion	Excessive Erosion	Continuing Erosion
	$D_{95}B$ (mm)	FC (%)	f-m Sand (%)	$D_{15}F$ (mm)	$D_{15}F$ (mm)	$D_{15}F$ (mm)
Coarse	4.0	35	39	0.7	5	36
Average	3.5	42	39	0.7	5	32
Fine	2.0	61	28	0.7	2	18

- Plot the erosion boundaries on the original filter gradation curves (and the adjusted filter gradation curves for segregation or washout) on the D_{15} line. The erosion boundaries are shown on Figure D-6-H-5 for the original filter gradation and Figure D-6-H-6 for the adjusted filter gradation after segregation or washout.

- Estimate the proportion of the original filter gradation (and filter gradation after segregation or washout) within each of the erosion categories for the coarse, average, and fine base soil gradations. By inspection, the approximate proportions of the gradation band within each erosion boundary are summarized in Table D-6-H-9. The proportions for the no erosion category are calculated below.

For the original filter gradation:

Coarse base soil gradation: $P_{NE,\ coarse} = 1 - (0 + 0.30 + 0.60) = 0.10$

Average base soil gradation: $P_{NE,\ average} = 1 - (0 + 0.30 + 0.60) = 0.10$

Fine base soil gradation: $P_{NE,\ fine} = 1 - (0.05 + 0.45 + 0.40) = 0.10$

For the adjusted filter gradation after segregation or washout:

Coarse base soil gradation: $P_{NE,\ coarse} = 1 - (0.02 + 0.58 + 0.40) = 0$

Average base soil gradation: $P_{NE,\ average} = 1 - (0.04 + 0.56 + 0.40) = 0$

Fine base soil gradation: $P_{NE,\ fine} = 1 - (0.10 + 0.80 + 0.10) = 0$

Table D-6-H-9 Proportions for Example Filter Material

Base Soil Gradation	NE	SE	EE	CE
Original Filter Gradation				
Coarser (10%)	0.10	0.60	0.30	0.00
Average (80%)	0.10	0.60	0.30	0.00
Finer (10%)	0.10	0.40	0.45	0.05
Adjusted Filter Gradation after Segregation or Washout				
Coarser (10%)	0.0	0.40	0.58	0.02
Average (80%)	0.0	0.40	0.56	0.04
Finer (10%)	0.0	0.10	0.80	0.10

- Make an initial estimate of the probabilities of no erosion, some erosion, excessive erosion, and continuing erosion by calculating the sum-product of the percentage of base soil gradations and the estimated percentage of no erosion, some erosion, excessive erosion, and continuing erosion for the coarse, average, and fine base soil gradations. The calculations are as follows, where N corresponds to the representative base soil gradation (i.e., as a percentage of all gradation tests) and n = (100 –N)/2 corresponds to the percentage finer or coarser of the base soil:

For the original filter gradation:

P_{NE} = (10/100)(0.10) + (80/100)(0.10) + (10/100)(0.10) = 0.1

P_{SE} = (10/100)(0.60) + (80/100)(0.60) + (10/100)(0.40) = 0.58

P_{EE} = (10/100)(0.30) + (80/100)(0.30) + (10/100)(0.45) = 0.315

P_{CE} = (10/100)(0.00) + (80/100)(0.00) + (10/100)(0.05) = 0.005

For the adjusted filter gradation after segregation or washout:

P_{NE} = (10/100)(0.00) + (80/100)(0.00) + (10/100)(0.00) = 0

P_{SE} = (10/100)(0.40) + (80/100)(0.40) + (10/100)(0.10) = 0.37

P_{EE} = (10/100)(0.58) + (80/100)(0.56) + (10/100)(0.80) = 0.586

P_{CE} = (10/100)(0.02) + (80/100)(0.04) + (10/100)(0.10) = 0.044

- The probability of continuation without considering other factors (e.g., filter thickness, continuity of coarse zones, presence of a free face, etc.) could be estimated on the low side as the probability of continuing erosion or 0.005 for the original filter gradation and 0.044 for the adjusted filter gradation after segregation or washout. If it were judged that there was a 10 percent chance of the segregated or washed out filter being in contact with the core, the minimum probability of continuation could be estimated as 0.1(0.044) + 0.9(0.005) ≈ 0.01. The maximum probability of continuation is based on examining the excessive and some erosion boundaries in Figures D-6-H-6 and D-6-H-7. For example, if it were judged that there was about a 50 percent chance that soil within the excessive erosion category would not eventually plug off but practically no chance that soil within the some erosion category would not plug off, then the probability of continuation for the original filter gradation could be estimated as 0.005 + 0.5(0.315) ≈ 0.16. Similarly, the probability of continuation for the adjusted filter gradation could be estimated as 0.044 + 0.5(0.586) ≈ 0.34. The weighted maximum probability of continuation is 0.1(0.34) + 0.9(0.16) ≈ 0.2. Therefore, probability of continuation is ranges from about 0.01 to 0.2.

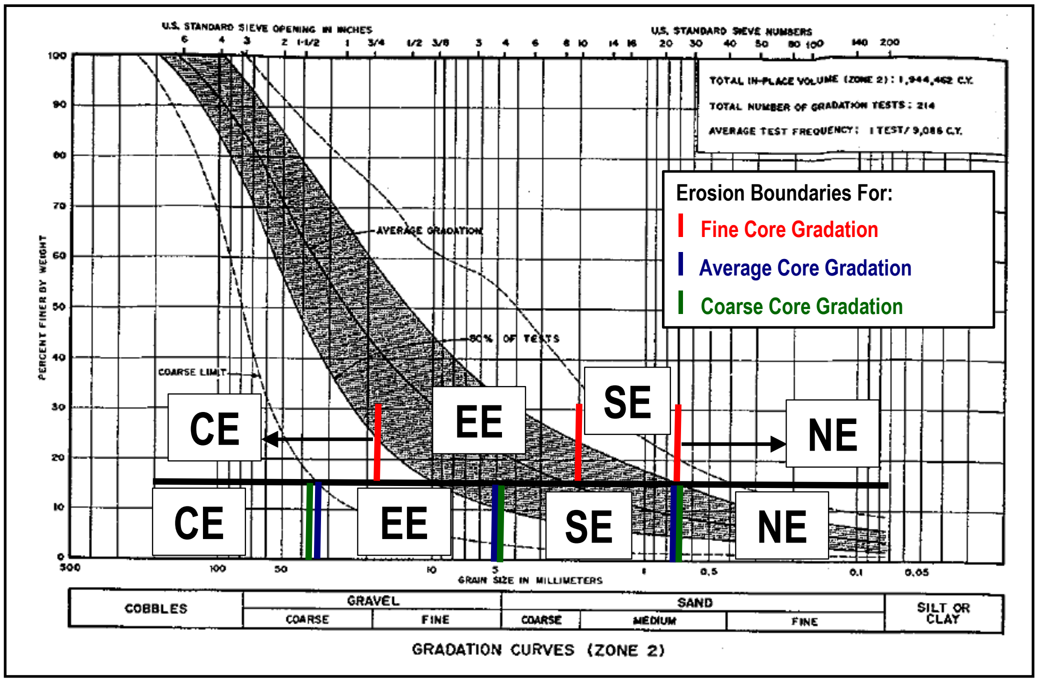


Figure D-6-H-5 Erosion Boundaries on D15F of Original Filter Gradation

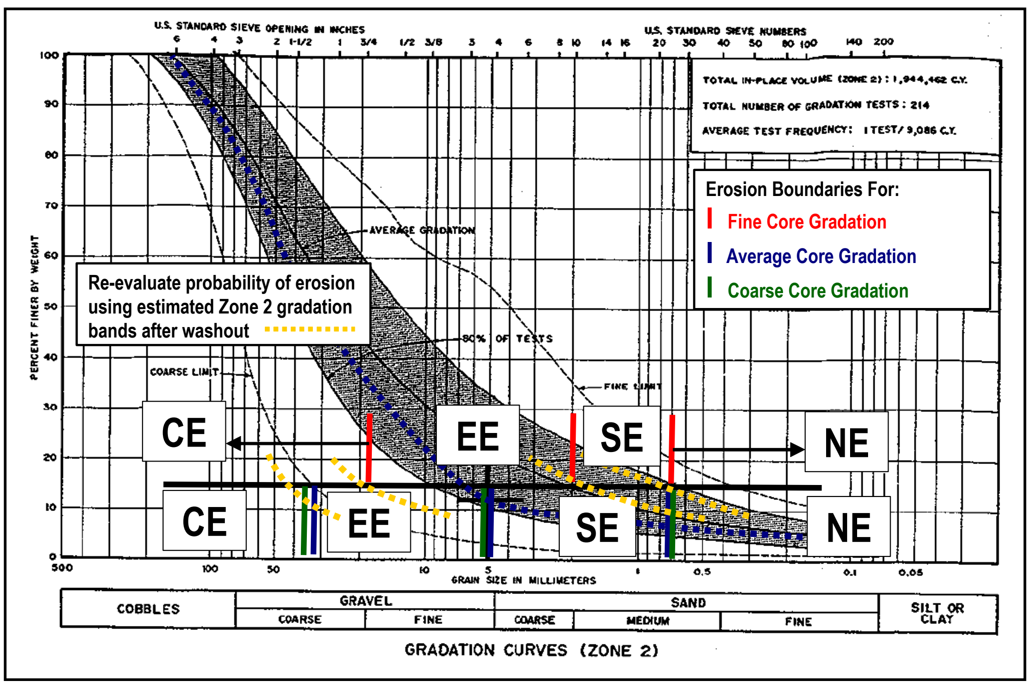


Figure D-6-H-6 Erosion Boundaries on D15F of Adjusted Gradation after Segregation or Washout

Appendix D-6-I: Rate of Enlargement of a Pipe

While the resistance to initiation of concentrated leak erosion is characterized by the critical shear stress, the rate of pipe enlargement in the progression phase under a significantly long loading event is characterized by the erodibility coefficient (rate of change of erosion rate). There are several methods for estimating the erosion properties of soils for concentrated leak erosion. The Hole Erosion Test (HET), Jet Erosion Test (JET), and Erosion Function Apparatus (EFA) are the most widely used tests. Further details on methods to estimate the erodibility parameters are discussed in Chapter D-1 Erosion of Rock and Soil.

Figure D-6-I-1 illustrates the importance of soil erodibility (characterized by the Hole Erosion Test index) on the time for erosion to progress, based on the following assumptions: unrestricted potential for erosion (i.e., no flow limitation, continuing erosion condition); initial pipe diameter of 25 mm (1 inch); zero critical shear stress (which is conservative, particularly for $I_{HET} > 3.5$); shape of pipe remains circular; pipe can sustain a roof while it enlarges; and reservoir level remains constant. The time to erode to 2 m in diameter is about 20 percent greater.

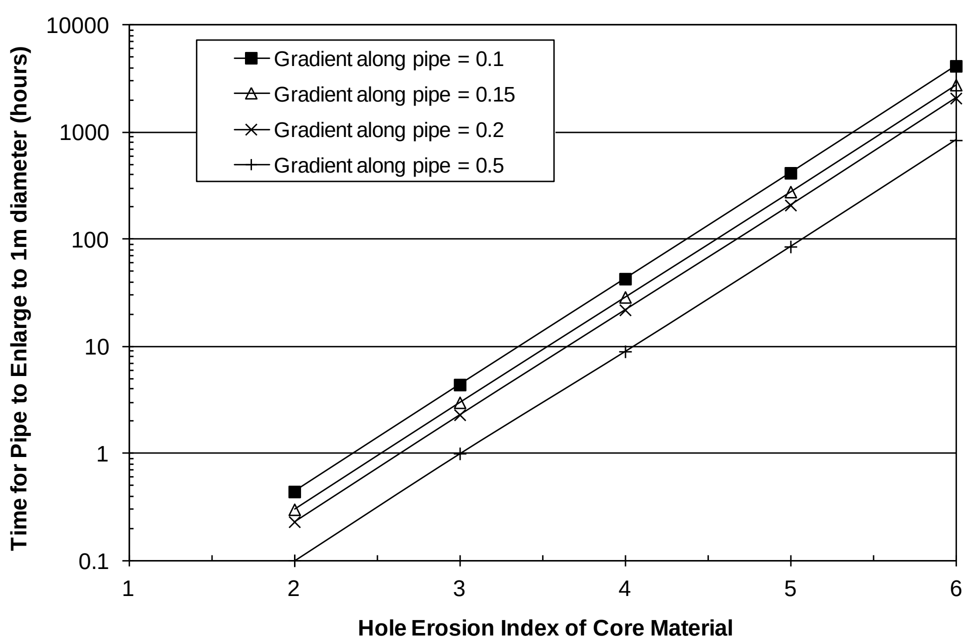


Figure D-6-I-1 Approximate Time for a Pipe to Enlarge from 25 mm to 1m in Diameter (Fell et al. 2008)

The rate of enlargement of a pipe in the progression phase can be estimated using methods described in Wan and Fell (2002) for a circular pipe. The rate of erosion per unit surface area at time t is given by:

$$\dot{\varepsilon}_t = \frac{1}{\psi_t}\frac{dV_t}{dt} = k_d(\tau - \tau_c) \text{ for volume erosion} \qquad \textbf{Equation D-6-I-1}$$

$$\dot{m}_t = \frac{1}{\psi_t}\frac{dM_t}{dt} = C_e(\tau - \tau_c) \text{ for mass erosion} \qquad \textbf{Equation D-6-I-2}$$

where $\Psi_t = P_{w,t}\, L$ = surface area of the pipe at time t; dV_t/dt = rate of soil volume removal due to erosion at time t; dM_t/dt = rate of soil mass removal due to erosion at time t; τ = hydraulic shear stress for the reservoir level under consideration; τ_c = critical shear stress for initiation of erosion; and k_d = erodibility coefficient; and C_e = coefficient of soil erosion.

Using the above equations, the erosion loss (per unit length) can be rewritten as:

$$dV_t = \dot{\varepsilon}_t \psi_t dt = k_d(\tau - \tau_c)P_w dt = k_d(\tau - \tau_c)(\pi\phi_t)dt \qquad \textbf{Equation D-6-I-3}$$

for volume erosion

$$dM_t = \dot{m}_t \psi_t dt = C_e(\tau - \tau_c)P_w dt = C_e(\tau - \tau_c)(\pi\phi_t)dt \qquad \textbf{Equation D-6-I-4}$$

for mass erosion

The change in pipe diameter at time t is given by:

$$d\phi_t = 2\frac{dV_t}{\pi\phi_t} \text{ for volume erosion} \qquad \textbf{Equation D-6-I-5}$$

$$d\phi_t = 2\frac{dM_t}{\rho_d\pi\phi_t} \text{ for mass erosion} \qquad \textbf{Equation D-6-I-6}$$

These equations can be readily setup in a spreadsheet to estimate the pipe diameter for user-specified time increments or steps based on estimates of hydraulic shear stress and erodibility parameters previously described and the following assumptions:

- Linear head loss from upstream to downstream
- Steady uniform flow along the pipe
- Zero pressure head at the downstream end
- Shape of the pipe remains circular
- Enlarging pipe can sustain a roof
- Uniform frictional resistance along the surface of the pipe or crack
- Driving force = frictional resistance
- Reservoir remains constant with time

An example of portrayal of analytical results is shown in Figure D-6-I-2. In this example, an initial pipe diameter was assumed, and the critical shear stress, erodibility coefficient, and range of pipe diameter at failure (D_f) were estimated by a risk team during an elicitation. Based on the estimated pipe diameter as a function of time, this figure can be used to help develop a list or more likely and less likely factors for the potential time available for intervention or full breach development as a function of reservoir level.

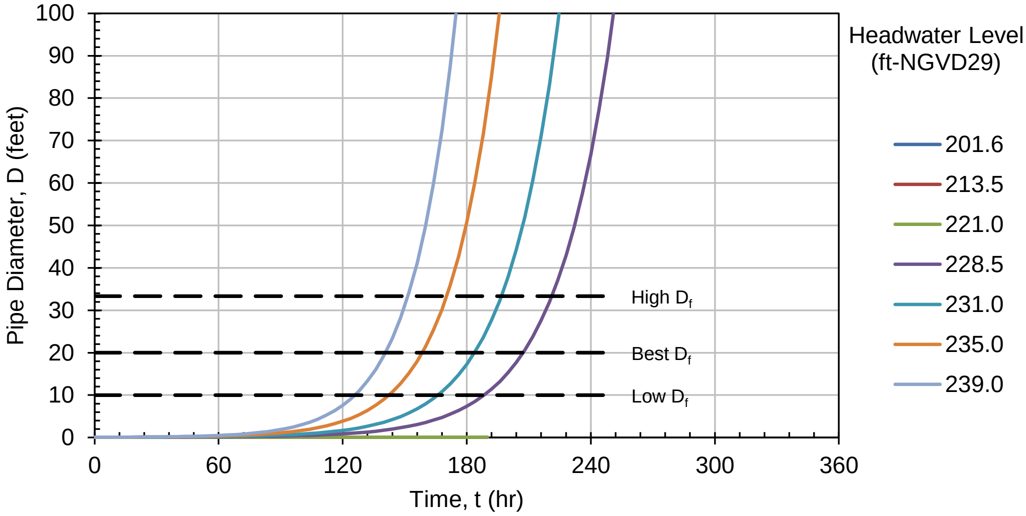


Figure D-6-I-2 Example Portrayal of Analytical Results for Rate of Enlargement of a Pipe

Appendix D-6-J: Tables of More and Less Likely Factors for Different Categories of Internal Erosion